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A STUDY OF THE I-15 RECONSTRUCTION PROJECT TO INVESTIGATE VARIABLES AFFECTING BRIDGE DECK CRACKING

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UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

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16. Abstract

Bridge deck cracking is a national problem and many states have undertaken studies to determine the causes and identify possible solutions. Field observations performed by UDOT personnel between April 1997 and August 2001 revealed the presence of cracks on the top and underside of many new I-15 bridge decks. The type and amount of cracking observed on these new structures was similar to what has been experienced on other bridges nationwide. This study has been undertaken in an attempt to identify the factors contributing to bridge deck cracking and to ascertain methods and procedures to minimize the problem on future projects. With the large number of I-15 bridges constructed in such a short period of time, Utah has a unique opportunity to isolate and identify variables that contribute to bridge deck cracking. Some key variables include the size of concrete placements, continuity of multi-span decks, concrete curing procedures, post-tensioning strain differentials, restraint between decks and other bridge elements, and deck placement sequencing. Bridge decks with minimal cracking can be built. This study points out numerous causes of concrete bridge deck cracking and suggests ways to minimize it.

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1. INTRODUCTION

The I-15 Reconstruction Project of the Utah Department of Transportation (UDOT) began in April 1997 and was completed during the summer of 2001. This project was, to date, the single largest interstate reconstruction project ever attempted in the state of Utah. The reconstruction included the demolition and rebuilding of approximately 27 km (16.8 miles) of urban interstate, at a total cost of approximately $1.5 billion. It involved the placement of roughly 3.8 million cubic meters (5.0 million cubic yards) of embankment fill and the construction of nearly 160 retaining walls. Many bridge structures were completely demolished for the project and 142 new bridge structures were constructed.

Bridge deck cracking is a national problem and many states have undertaken studies to determine the causes and identify possible solutions. Field observations performed by UDOT personnel revealed the presence of cracks on the top and underside of many new I-15 bridge decks. The type and amount of cracking observed on these new structures was similar to what has been experienced on other bridges nation wide.

Deck cracking does not generally represent any load carrying inadequacy, but poses a potential long term durability issue for the bridge. Bridge decks have steel reinforcing bars buried within the concrete slab. The concrete and steel work together, with composite action, to resist the applied loads. The concrete is designed to work in compression and the steel is designed to work in tension. This composite action allows the cracked concrete slabs to continue resisting applied loads on condition that the steel reinforcement is not compromised.

However, cracked concrete bridge decks are a concern. Cracks allow water to penetrate the concrete slab. This water often transports chlorides and other salts and puts them in contact with the reinforcing steel. Steel oxidizes when in contact with water and oxygen, particularly in the presence of chlorides. Long term, unchecked corrosion of mild steel reinforcement and post-tensioning strands in bridge decks would become a concern. Not only does severely oxidized steel have reduced capacity to carry tensile loads, the process of oxidation increases the volume of steel which creates tensile stresses in the adjacent concrete. Concrete has limited tensile strength and therefore cracks further. This in turn exposes more steel to the oxidizing agents and the process repeats itself until much of the concrete is cracked and spalled. Once a significant percentage of the reinforcement has corroded or a considerable portion of the concrete has spalled off, the reinforced concrete bridge deck must be repaired or replaced. Additionally, if any water freezes inside the cracks, the resulting expansion may spall the concrete.

2. PURPOSE OF THIS STUDY

This study has been undertaken in an attempt to identify the factors contributing to bridge deck cracking and to ascertain methods and procedures to minimize the problem on future projects. With the large number of I-15 bridges constructed in such a short period of time, Utah has a unique opportunity to isolate and identify variables that contribute to bridge deck cracking. Along with traditional techniques, some methods never before used by UDOT were implemented throughout the bridge design and construction phases of the I-15 Reconstruction Project. Some of the new techniques include:

- The use of silica fume concrete. All of the cast-in-place bridge deck concrete used in the I-15 Reconstruction Project had silica fume (5% by weight of cementitious materials) added to it. This material is generally added to increase the strength and density of concrete.

- The use of precast concrete deck panels. The majority of new concrete girder bridges were constructed with precast concrete deck panels. These panels serve as stay-in-place formwork and constitute the lower portion of the bridge deck. The remaining upper portion of the bridge deck consists of a traditional cast-in-place, reinforced concrete slab that becomes composite with the lower precast panels.

- The use of wide-spaced steel girders together with transversely post-tensioned concrete decks.

- The use of deep, long span, spliced, post-tensioned concrete girders. These girders were erected in three separate sections on temporary supports. Once the girders were spliced and the deck construction was complete, the girder sections were longitudinally post-tensioned and the interior temporary supports were removed.

This study provided an opportunity to evaluate many traditional methods of bridge deck construction and design as well as these new techniques. Multiple variables were evaluated in an attempt to determine any contribution that they may make towards bridge deck cracking. Given the ever increasing widths and span lengths of modern bridges it is unlikely that deck cracking can be completely eliminated. However, this study attempts to identify the major contributing factors and recommend methods for crack reduction.

2.1 Cracking Mechanisms

There are many factors that may contribute to the cracking of concrete bridge decks. According to Phillips et al. (1997), restrained shrinkage of the concrete is the most common cause. This restraint is typically provided by rigid attachment to large masses such as abutments, bents, girders, and diaphragms. Standard portland cement concrete shrinks as it cures, and this shrinkage may be compounded by improper placement and curing procedures. Improper curing leaves the concrete surface vulnerable to rapid water loss, which results in rapid shrinkage and cracking. Although difficult to completely separate one factor from another, Issa (1999) suggests the top ten causes of bridge deck cracking are (listed in descending order of importance):

1. Inadequate concrete curing procedures which result in high evaporation rates and thus a high magnitude of shrinkage, especially in early age concrete.
2. The use of high slump concrete.
3. High water-to-cement ratios due to inadequate mixture proportions and retempering of concrete.
4. Insufficient top reinforcement cover.
5. Inadequate vibration of the concrete.
6. Deficient reinforcing details of the joint between a new and old deck.
7. Sequence of deck section placement.
8. Vibration and loads from machinery.
9. The weight of concrete forms.
10. The deflection of formwork.

In addition to these common factors, there are other design and maintenance issues that may also contribute to cracking.

2.2 Crack Avoidance

In order to increase bridge durability, it is vital to minimize bridge deck cracking. Eliminating cracks in concrete is extremely difficult and generally not possible. However, with careful design and construction practices, the amount of cracking may be minimized. When there is no way to eliminate the formation of cracks, other measures should be taken to reduce the potential for reinforcing steel oxidation.

2.2.1 Design Issues

Careful consideration of potential cracking mechanisms during the design phase of a bridge structure will improve deck performance and minimize cracking. These issues include:

- Concrete mix design. The water-to-cement ratio of a mix and the use of water reducing agents should be considered by the designer. Concrete admixtures are used to improve concrete performance. There are disadvantages, however, that should be considered when some admixtures are specified. For example, silica fume is added to increase the strength and density of a concrete mix. However, silica fume concrete has high finishing demands and is particularly sensitive to wet curing.

- Restraint. The type and timing of the connection of bridge decks to rigid structural elements such as abutments, bents, and diaphragms must be carefully considered. The proper use of expansion joints will relieve some restraint within the concrete bridge deck. However, it is important that creep, temperature, and shrinkage are all taken into account when determining expansion joint specifications (Chowchuvech and Gee, 2003). Additionally, expansion joints compound the seismic design of a bridge and oftentimes provide a path for salt laden water to drip on the supporting beams and columns.

- Girder type. There are many types of girders available for use in bridges. The girder types evaluated in this study are:

    - Simply-supported, precast, pre-stressed concrete girders.
    - Spliced, post-tensioned, precast concrete girders.
    - Steel, I-shaped girders.

  A designer should consider the various characteristics (e.g., span-to-depth ratio, stiffness, thermal characteristics, etc.) of each possible girder type. Consideration should also be given to single-span design versus continuous-span design.

- Girder spacing. Since bridge decks are designed to span between girders, the span-to-depth ratio of a concrete deck may influence longitudinal cracking. The amount and type of top and bottom reinforcement required in the deck also depends on the girder spacing.

- Concrete shrinkage. A bridge deck design should anticipate the amount of concrete shrinkage that will take place. Calculations should be performed to define the limits on the size of any given deck placement. Timing between placements and connection details between the slab and other components should also be addressed. Pourback strips may reduce shrinkage cracking by reducing the amount of restraint that the deck is exposed to at early ages.

- Other issues that a designer should consider include bridge skew, slope and drainage of the bridge deck, closed piping systems to keep the drainage water away from the bridge superstructure, etc.

2.2.2 Construction Issues

A well designed bridge structure may still develop significant deck cracking if the selected construction methods do not consider the structure as a whole. Care must be taken to conform to the design details, sequences, and specifications. Other important issues during the construction phase of a bridge deck include:

- Maintaining adequate cover for top reinforcing bars.

- Workability of the concrete. Do not add more water than specified to the concrete mix.

- Consideration of atmospheric conditions. Extreme temperatures, hot or cold, require special consideration during concrete placement. If a newly placed concrete surface is exposed to wind and heat, the evaporation of the curing water will happen at a higher rate. Conversely, exposure to extremely cold temperatures will cause the water to freeze within the newly placed concrete.

- Supporting of the formwork. All formwork should be well constructed and supported to minimize the deflections and displacements of the newly cast concrete. The stiffness and stability of temporary supports are particularly critical.

- Concrete must be properly cured. This requires that adequate moisture be provided to the fresh concrete throughout the cement hydration process.

2.2.3 Maintenance Issues

Concrete bridge decks are very expensive to repair and replace. Good maintenance practices will extend the life of any bridge deck. A good maintenance program would include:

- Annual wash-downs after the winter season. This is particularly important in regions where de-icing salts are applied directly to the bridge decks.

- Crack sealing. Cracks that allow deep water penetration should be dealt with as they are discovered. This would require careful inspections of the bridge decks at regular intervals. Routing and epoxying cracks will reduce water and salt ingress into the deck.

- Concrete sealers. If the decks are uncracked, a good concrete sealer may be applied to keep salts from migrating into the deck. The effective life span of sealers may vary significantly and should be considered. Lightweight, impermeable bridge deck overlay systems could also be implemented to cover modest cracks and keep water out of the decks.

3. SCOPE OF THIS STUDY

The scope of this study consists of the following:

- Selection of bridges to study;
- Literature review;
- Database design;
- Collect data and populate database;
- Data analysis;
- Establish conclusions and recommendations.

3.1 Bridge Selection

During the I-15 Reconstruction Project, 142 new bridges were constructed. There are four major categories of new bridges:

- SPT - Steel girders with transversely post-tensioned concrete deck
- SC - Steel girders with reinforced concrete deck
- PC - Pre-stressed concrete girders with reinforced concrete deck
- SPC – Spliced, post-tensioned concrete girders with reinforced concrete deck.

The 71 bridges, listed in Table 1, were selected for analysis because of the safe and relatively easy access to the bridge undersides. In general, bridges at major interstate junctions were avoided for reasons of safety and the expense and inconvenience of closing interstate traffic lanes to allow for proper inspection. The 71 selected bridges represent 50% of the total population of new bridge structures along the I-15 corridor through Salt Lake City. This sample set was established to represent the typical bridge type that will most likely be constructed by UDOT in the near future. For this study, the new bridges were identified by their RFP (Request For Proposal) numbers. These numbers were assigned to each structure during the environmental analysis process and were used in the preliminary design and construction of each bridge. Once a structure was completed and turned over to UDOT, a new number (the UDOT Inventory Number) was assigned.

Table 1 - A list of the 71 bridges studied along with the corresponding structure type and bridge location.

RFP Bridge No.	Bridge Type	Bridge Location	RFP Bridge No.	Bridge Type	Bridge Location
3	PC	I-15 SB over 10000 S	145	SPT	I-15 NB over Andy Ave
3.5	PC	I-15 NB over 10000 S	147	SPT	I-15 SB over Andy Ave
8	SPC	I-15 NB over 9000 S	148	PC	I-15 NB CD over 1700 S
10	SPC	I-15 SB over 9000 S	149	SPT	I-15 NB CD over Andy Ave
12	PC	I-15 NB over Wasatch St (8000 S)	150	PC	I-15 SB CD over 1700S
14	PC	I-15 SB over Wasatch St (8000 S)	151	SPT	I-15 SB to I-80 EB over Andy Ave
16	PC	I-15 NB over 7800 S (Center St)	152	PC	I-15 NB over 1700 S
18	PC	I-15 SB over 7800 S (Center St)	153	PC	I-15 SB CD over Andy Ave
20	SPC	I-15 NB over 7200 S	154	PC	I-15 SB over 1700 S
22	SPC	I-15 SB over 7200 S	155	PC	I-15 SB CD to I-15 SB over Andy Ave
23	SPT	I-15 NB / I-215 WB Ramp over 7200 S	156	PC	I-15 900S-A over 1300 S
26	SC	I-15 NB over UPRR	158	PC	I-15 SB CD over 1300 S
27	SC	I-215 EB to I-15 SB Ramp over UPRR	160	PC	I-15 NB over 1300 S
28	SC	I-15 SB over UPRR	162	PC	I-15 SB over 1300 S
29	PC	I-215 EB to 7200 S / I-15 SB CD Ramp over UTA RR	168	PC	I-15 NB over 500 W and UPRR
30	SPC	I-15 NB over UTA RR	170	PC	I-15 SB over 500 W and UPRR
32	SPC	I-15 SB over UTA RR	174	SPT	I-15 NB over 900 S
50	SC	I-15 NB over 5900 S	176	SPT	I-15 SB over 900 S
52	SC	I-15 SB over 5900 S	180	PC	I-15 NB over 800 S
54	SPC	I-15 NB over 5300 S	182	PC	I-15 SB over 800 S
56	SPC	I-15 SB over 5300 S	196	SPT	I-15 NB over 400 S
60	PC	I-15 NB over 4800 S	198	SPT	I-15 SB over 400 S
62	PC	I-15 SB over 4800 S	200	PC	Ramp NW over 400 S
64	SPC	I-15 NB over 4500 S	202	SPT	Ramp ES over 400 S
66	SPC	I-15 SB over 4500 S	212	PC	I-15 NB over 200 S
70	PC	I-15 NB over UPRR	214	PC	I-15 SB over 200 S
72	PC	I-15 SB over UPRR	216	SPT	I-15 NB over S Temple
74	SPC	I-15 NB over 3300 S	218	SPT	I-15 SB over S Temple
76	SPC	I-15 SB over 3300 S	220	PC	I-15 NB over N Temple
112	SPT	I-80 WB (Ramp) To SR 201 WB over UTA RR	222	PC	I-15 SB over N Temple
114	SPT	I-80 WB over UTA RR	224	PC	I-15 NB over 300 N
116	SPT	I-80 EB over UTA RR	226	PC	I-15 SB over 300 N
138	PC	I-15 NB CD over 2100 S	230	SC	Ramp SW over 200 S
140	PC	I-15 NB over 2100 S	702	SPT	I-15 SB CD over 7200 S
142	PC	I-15 SB CD over 2100 S	12002	PC	I-15 SB CD over 2100 S
144	PC	I-15 SB over 2100 S			

As shown in Table 2, all four bridge categories are well represented by the set of 71 bridges selected for this study. Of these 71 bridges, 16 are SPT, 6 are SC, 37 are PC, and 12 bridges are SPC. The sample set and the total population of new bridges contain similar percentages of the four bridge types.

Table 2 - Comparison of the population of new bridges and the sample set of bridges considered for this study.

Structure Type	Number of Bridges in Total I-15 Population	% of Total I-15 Population	Number of Bridges in Sample	% of Sample
SPT	36	25.3	16	22.5
SC	24	16.9	6	8.5
PC	63	44.4	37	52.1
SPC	16	11.3	12	16.9
Combination	3	2.1	0	0.0

3.2 Literature Review

A comprehensive literature review concerning bridge deck cracking, silica fume concrete, concrete creep, shrinkage in concrete, and bridge design considerations was undertaken to help gain an understanding of what is happening in other states. Along with the few references in Section 2, Section 4 provides a summary of the literature review.

3.3 Database Design

This study required the development of a database so that the characteristics and specific information pertaining to each structure could be analyzed, along with measures of their relative cracking. Microsoft Access software was chosen for this application because of its availability and ease of use. The database consists of many fields by which the data can be sorted. These fields include bridge characteristics such as type of construction, deck size, date of deck placement, etc. Included in Section 7 is a complete listing of all database fields.

3.4 Collect Data and Populate Database

In order to facilitate the requirements of this study, data were collected from numerous sources. A review of the structural plans of all 71 bridges was conducted. From these plan reviews, characteristics of each bridge (e.g., size, shape, design criteria, etc.) were recorded.

Data from observations made by UDOT personnel prior to the beginning of this study were included in the database for all 142 bridges. For this study, field observations of the underside of each bridge in the sample set (71 bridges) were made between December 2002 and August 2003. A limited number of deck topside inspections were also completed in August 2003. Throughout this inspection process, hundreds of digital photographs were taken to document the bridge observations. The majority of the underside observations were conducted during inclement weather to facilitate the viewing of any water that might be permeating through the bridge decks.

3.5 Data Analysis

Once completely populated, the database was queried many different ways. Query combinations that might reveal dominant variables were investigated. These queries were performed in an attempt to determine trends indicating causative factors contributing to specific cracks. Refer to Appendix B for a sample set of database queries.

3.6 Conclusions and Recommendations

Comparison of the information obtained from the literature review and the data analysis led to a better understanding of why bridge deck slabs crack. These conclusions and recommendations are contained in Section 10.

4. LITERATURE REVIEW

Cracked bridge decks are of major concern for transportation departments all across the country (Yunovich and Thompson, 2003; Xi et al., 2003, Petrou et al., 2001). In a 1998 report published by the American Society of Civil Engineers (ASCE), the overall condition of bridge structures in the United States was rated as 'poor.' According to the report, 15% of all bridges in the National Bridge Inventory are structurally deficient. The estimated total cost to replace these deficient bridges is approximately $29 billion (Yunovich and Thompson, 2003). A large amount of these deficiencies were caused by corrosion which resulted from water reaching the reinforcing steel within the concrete of the bridge. Much has been written on this subject over the years.

Hadidi and Saadeghvaziri (2003) conducted a comprehensive literature review to gain an understanding of why transverse cracking is common in bridge decks. Early age transverse deck cracking has been observed in most geographical locations. The full depth, regularly spaced cracks have been known to widen with time. Oftentimes transverse cracks occur over transverse reinforcing and increase with an increase in reinforcement bar size. According to Hadidi and Saadeghvaziri (2003), many studies indicated that concrete bridge decks on steel girders had higher tendencies to crack when compared to decks on concrete girders. Furthermore, continuous spans exhibit a higher amount of cracking when compared to single spans.

The Colorado Department of Transportation (CDOT) recently released a report outlining the condition of bridge decks in Colorado (Xi et al., 2003). Currently, 82% of Colorado's bridge decks have cracking problems. Xi et al. (2003) categorize the factors that cause cracking in newly constructed bridge decks as material, design, construction, and environmental factors. Some of the recommendations from this report are:

- to limit silica fume to 5% by weight of cementitious material;

- to use smaller sized reinforcement in the negative moment areas of bridge decks;
- to give preference to concrete girders because of the equivalent coefficients of thermal expansion between girder and deck;
- to promptly seal all cracks that develop, particularly within the first year after bridge deck placement;
- to decrease longitudinal restraint on bridge decks wherever possible.

Many of these factors have been previously recognized (Hadidi and Saadeghvaziri, 2003; Whiting and Detwiler, 1998) as important issues to consider when designing or constructing concrete bridge decks.

4.1 Concrete Shrinkage and Creep

Concrete experiences different types of shrinkage. Autogeneous shrinkage occurs as the cement hydrates. The chemical reactions that occur during hydration create a change in volume of the overall system. Drying shrinkage is the volume change that results from the loss of moisture from the concrete to the environment (ACI Committee 209, 1997). Plastic shrinkage is a special form of drying shrinkage in which the loss of moisture occurs rapidly from the concrete surfaces while it is still in a plastic state. Concrete shrinkage of all types results in a reduced volume and the development of internal tensile stresses which cause cracks.

"The time dependant increase of strain in hardened concrete subjected to sustained stress is defined as creep" (ACI Committee 209, 1997). Controversy has arisen over the best method to calculate creep. This stems from the fact that there is no distinct separation between instantaneous strain and time dependant strain. Additionally, the term creep encompasses both drying creep and basic creep. "Basic creep occurs under conditions of no moisture movement to or from the environment" and "drying creep is the additional creep caused by drying" (ACI Committee 209, 1997). These conditions make creep estimations difficult.

Unlike shrinkage, creep has a positive effect on early aged concrete. As concrete creeps the tensile stresses in the concrete caused by the drying shrinkage are relaxed and therefore the risk of cracking is reduced.

4.2 Shrinkage Compensating Cement

The use of Type K shrinkage compensating concrete can help offset some of the early shrinkage caused by the use of silica fume (Whiting and Detwiler, 1998; ACI Committee 223, 1998). As with most concrete admixtures, however, there are disadvantages associated with using this type of cement. Shrinkage compensating concrete requires more attention to detail than does regular concrete for mixing and placing procedures. This puts extra importance and requires more effort on the part of the contractors (Phillips et al., 1997).

The Ohio Turnpike Commission may be the greatest user of shrinkage compensating concrete in the United States and has approximately 520 bridges in operation that were constructed with this type of concrete. The Commission has had a positive experience with this type of concrete and claim that it has greatly mitigated shrinkage cracking (Phillips et al., 1997). According to Phillips et al. (1997), the American Concrete Institute (ACI) recommends that shrinkage compensating concrete not be used in bridge decks on concrete girders because "they are felt to present excessive external restraint to longitudinal expansion and shrinkage-compensating action." This is likely due to the significant stiffness against out-of-plane bending that large concrete girders exhibit.

4.3 Silica Fume Concrete

Silica fume concrete was first used in highway applications in the United States in the mid-1980s (Whiting and Detwiler, 1998). Since that time, the use of silica fume concrete has grown considerably. As outlined below, it is generally understood that silica fume concrete must be properly cured to prohibit cracking. However, there is some disagreement as to the optimal dosage of silica fume. Also, the amount of concrete cracking that is directly related to the use of silica fume is not fully understood.

Silica fume, or microsilica, is a byproduct of silicon-metal production. Silica fume is much finer grained than portland cement. The average silica fume particle diameter is 0.1 μm (0.004 mils). Silica fume generally contains over 90% silicon dioxide and has a specific gravity in the range of 2.10 to 2.55. During cement hydration, silica fume reacts with free lime in the concrete to form a strong cementitious compound called Calcium Silicate Hydrate (Lafave et al., 2002). This compound fills the interstitial spaces between the cement paste matrix and the aggregate particles. Due to the resulting dense particle matrix, the concrete has a low bleeding rate. As the cement begins to hydrate small capillaries within the concrete matrix dry very quickly and cause capillary tension. At this plastic stage, the concrete is not strong enough to resist the tensile stresses and therefore cracks form. However, if an external source of water is available to replenish the capillaries at the same rate the water is adsorbed or lost to evaporation, no capillary tension is produced (Morin et al., 2002).

According to ACI Committee 234 (1996), as the amount of silica fume in a concrete mix is increased, the water demand of the mix also increases. It is recommended that silica fume concrete be made with a water-reducing admixture, a high-range water-reducing admixture (HRWRA), or both (ACI Committee 234, 1996). Because silica fume concrete has an increased tendency to develop early-age plastic shrinkage cracks, care must be exercised to limit the amount of moisture that is lost at early ages (ACI Committee 234, 1996). ACI Committee 234 (1996) recommends the use of evaporation retarders or fogging as a way to limit moisture loss. Fresh silica fume concrete is more cohesive than traditional concrete and may appear to become sticky. This makes it difficult to properly finish silica fume concrete. ACI Committee 234 (1996) states that "the best way to establish exact finishing methods for any particular project is to stage small trial placements prior to the start of the actual work." For the I-15 Reconstruction Project, UDOT used the construction of a number of temporary structures (including I-15 northbound over 7800 south and I-15 northbound over 8000 south) to facilitate this ACI recommendation.

Morin et al. (2002) recommends maintaining a moist environment during concrete placement by using fog sprays. They also suggest that curing membranes should not be used on high performance concrete because the membrane will prevent the penetration of curing water which is necessary to control autogenous shrinkage. It is critical to provide a period of moist curing for silica fume concrete in order to avoid shrinkage cracking. This period of moist curing must begin immediately after the concrete is placed and finished. If the concrete surface is allowed to dry before the water curing is applied, the menisci that rapidly form in the capillaries near the concrete surface may prevent future water from entering the concrete, thus defeating the purpose of wet curing (Morin et al., 2002).

Whiting and Detwiler (1998) performed a number of laboratory tests to determine the effects of silica fume on concrete cracking. The tests indicate that silica fume will not promote the cracking tendency of full-depth concrete mixes if the concrete has sufficient moist curing. They also found that the ultimate (long-term) shrinkage of silica fume concrete is no different from otherwise identical concretes without silica fume. Yet, due to the dense concrete matrix, silica fume does affect the time frame in which the shrinkage takes place. Early age shrinkage should be controlled by wet curing. At least seven full days of wet curing is required for silica fume concrete (Whiting and Detwiler, 1998). The drying shrinkage at later ages is more dominantly controlled by changes in the water-to-cement ratio of the concrete mix. Also, the tests of Whiting and Detwiler (1998) did indicate that shrinkage is more sensitive to changes in the water-to-cement ratio as the silica fume content is increased. They suggest that a dosage of microsilica between 6% and 8% is enough to effectively reduce chloride diffusion. Amounts of silica fume in excess of this range did not appear to be harmful; however, Whiting and Detwiler (1998) found that they are not cost effective.

Microsilica has been shown to retard chloride migration into concrete. This is due to the dense particle matrix of silica fume concrete. The tiny particles of silica fume fill the micro voids between the cement particles and aggregate. This dense matrix reduces the ability of water to propagate through the concrete, which provides added protection for the embedded steel reinforcement. Lafave et al. (2002) performed a comprehensive literature review of admixtures in structural concrete. It was found that doses of silica fume of at least 7% inhibited the corrosion of reinforcing steel in the concrete. They recommend a silica fume dosage between 10% and 15%. This dosage recommendation is significantly higher than the recommendation of others (Xi et al., 2003; Whiting and Detwiler, 1998). Lafave et al. (2002) also stress the importance of proper concrete curing in order to prevent early-age plastic and drying shrinkage cracking of silica fume concrete.

Kanstad et al. (2001) undertook a study to determine the effect of silica fume on early age crack sensitivity. Tests were performed on specimens ranging in silica fume content of 0 to 15%. The tests indicate that an increase in silica fume results in an increase in tensile strength and elastic modulus of the concrete. However, the tests also indicated that higher values of autogenous shrinkage accompany increases in silica fume content. Still, they found that the risk of cracking was affected more by a variation in water-to-cement ratio then by silica fume content.

As reported by Hadidi and Saadeghvaziri (2003), some studies show that without proper curing, the use of silica fume significantly increases cracking. A minimum moist curing period of 7 days is recommended with some agencies pushing to increase the period to 14 days. This curing period should begin immediately after deck placement.

4.4 Corrosion Inhibiting Additives

Another way of protecting concrete reinforcing steel is to use corrosion inhibiting concrete additives. There are two main types of these admixtures, those containing calcium nitrite and those containing amines and esters (Lafave et al., 2002). There is some debate in the literature about whether or not calcium nitrate reduces the rate of chloride ion diffusion into concrete. Calcium nitrite blocks the current path between adjoining reinforcement layers thereby shutting down the galvanic cell. Additionally, the nitrite reacts with the ferrous ions on the reinforcement surfaces to produce a passive ferric oxide protective film. This film protects the reinforcing bars from chloride ion attack. An adequate dose of calcium nitrite has been shown to reduce the corrodibility of reinforcement in good quality concrete without reducing the concrete strength (Lafave et al., 2002).

Amines and esters have been found to adversely affect some concrete properties. Concrete mixes with amine and ester admixtures have lower compressive strengths and less ability to entrain air (Lafave et al., 2002). These water-based admixtures are designed to develop an organic coating on the steel reinforcement and reduce the amount of chloride penetration into concrete. Some studies have shown amines and esters to be effective for steel reinforcement protection but other studies disagree (Lafave et al., 2002).

4.5 Material Properties

Schmitt and Darwin (1999) compared the material properties of concrete decks on 40 continuous steel girder bridges in the State of Kansas. “The results of the evaluation indicate that cracking in monolithic bridge decks increases with increasing values of concrete slump, percent of concrete volume occupied by water and cement, water content, cement content, and compressive strength, and decreasing values of air content (especially below 6.0%).” Upon completion of the study, Schmitt and Darwin (1999) recommend that not more than 27% of the total volume of bridge deck concrete consist of cement and water. They also recommend that bridge deck concrete have at least 6.0% air by volume in order to control cracking.

Silica fume concrete that is used in bridge decks typically has a water-to-cement ratio between 0.35 and 0.45 (ACI Committee 234, 1996). It is generally believed that higher water-to-cement ratios result in higher amounts of cracking due to larger amounts of shrinkage (Hadidi and Saadeghvaziri, 2003). Hadidi and Saadeghvaziri (2003) recommend a water-to-cement ratio for bridge deck concrete between 0.4 and 0.45 and even less if water reducing agents are used. However, Xi et al. (2003) recommend using a water-to-cement ratio not less than 0.40 but note that "an optimal water-to-cement ratio has yet to be determined."

4.6 Construction Practices

Construction practice has been found to play a major role in cracking of concrete bridge decks (Hadidi and Saadeghvaziri, 2003). Hadidi and Saadeghvaziri (2003) recommend the following guidelines for deck placement:

- If possible, within the limitation of the maximum placement length based on drying shrinkage considerations, place the entire deck at one time.
- For a bridge composed of simple spans that cannot be placed at one time, place each span in one placement.
- If multiple placements must be made for a multi-span continuous bridge, place the positive moment regions first and allow a 72 hour delay before placing the negative moment regions.
- Require priming of interfaced surfaces with an appropriate bonding agent when construction joints are created.
- Shore simply supported girders during deck construction.

For the construction of a new bridge deck in New Hampshire, the contractor was required to perform a "mock" placement of five cubic yards to ensure that the contractor and his workforce had the competence and ability to comply with the placement and curing specifications, particularly the time constraints. This also allowed for a trial batch of the specified concrete to be generated. This trial batching and trial pour optimized the future placement of the bridge deck (Waszczuk, 1999).

4.7 Precast Deck Panel Forms

The I-15 contractor chose to use precast concrete deck panels on most of the new concrete girder bridges. Laboratory tests have shown that precast concrete panel bridge decks are strong enough to resist typical design live loads. Abendroth (1995) constructed and tested 5 full scale models of this type of bridge deck. Each composite specimen consisted of two side-by-side 63.5 mm (2.5") thick precast panels with a 140 mm (5.5") thick reinforced concrete slab placed directly on top. One specimen was rectangular shaped in order to model a portion of a deck away from an abutment or bent. The remaining panels varied in trapezoidal shape so as to model the condition near an abutment for skewed bridges ranging from 0° to 40°.

All of the specimens were loaded according to AASHTO MS-18 truck loading. During service and factored loading, all specimens maintained full composite behavior. Abendroth (1995) found that during the ultimate loading tests, a crack always occurred through the topping slab directly above the panel joint. This is due to the rapid change in cross-section and the accompanying stress concentrations that develop at the panel joints. The smallest ultimate load was approximately 7 times the required service level load and the ultimate failure mechanism of all specimens was determined to be punching shear. Abendroth (1995) also found that the nominal strengths of the composite decks were not affected by skew angles of 15°, 30°, and 40°.

Fang et al. (1990) also conducted experiments designed to compare the qualities of full-depth cast-in-place concrete bridge decks and decks formed with precast concrete panels. These tests revealed that the deck formed with precast panels was stiffer, stronger, and more crack resistant than the traditional full-depth cast-in-place deck.

4.8 Reinforcement Protection

The ideal method to prevent the oxidation of steel reinforcement in concrete is to prohibit water from reaching it. This may be accomplished by maintaining adequate concrete cover over the reinforcing steel in an uncracked deck. Concrete cover serves as a layer of protection in keeping water and salts from reaching the reinforcing steel. If the concrete cover above the reinforcing steel is insufficient, the slab tends to crack directly above the reinforcing bars (Issa, 1999). Once the concrete above the reinforcement is cracked, it provides a pathway for water infiltration.

There are several types of corrosion resistant reinforcing steel that can be used in concrete bridge decks. A bridge designer should carefully consider each option when specifying special concrete reinforcing steel.

The use of epoxy coatings on concrete reinforcing steel is common throughout the United States. UDOT has been using epoxy coated reinforcing steel in bridge decks since the late 1980s and all of the bridge deck reinforcement used on the I-15 Reconstruction Project was epoxy coated. This epoxy coating is intended to protect the steel from any oxidizing agents that may penetrate the concrete. Epoxy coated reinforcement works well to inhibit steel oxidation providing the epoxy coating remains undamaged. Once the epoxy coating is damaged and the oxidizing agents contact the steel, the oxidation process will begin. This oxidation process will continue along the bar beneath the remaining epoxy coating (Better, 2001). Laboratory tests performed by Kahhaleh et al. (1993) show that even with damage to the epoxy coating, coated reinforcing bars outperformed uncoated bars in corrosion resistance. In fact, the worst epoxy coated bar tested by Kahhaleh et al. (1993) performed about 2.3 times better then the uncoated bars. The contractor on the I-15 Reconstruction Project was required to repair any observed damage to the epoxy coating on reinforcing steel before concrete placement.

Pyć et al. (2000) performed an extensive investigation into the field performance of epoxy-coated reinforcing steel in bridge decks for the Virginia Transportation Research Council. This study included inspection of about 250 concrete cores from eighteen bridge decks in Virginia ranging in age from 2 to 20 years. The cores were evaluated for "carbonation depth, moisture content, absorption, percent saturation, and chloride content at a 13-mm (0.5 in) depth." Additionally the epoxy coated reinforcement was visually inspected for damage, coating thickness, and adhesion. In all but one bridge deck older than four years, the epoxy coatings were debonding from the reinforcing bars. It generally takes longer than four years for chlorides to reach a sufficient level of concentration within the deck to initiate corrosion of reinforcing steel (Pyć et al., 2000). Figure 1 is a photograph of a typical joint reinforcing dowel that was removed from a concrete pavement slab after only a few years of service in Salt Lake City. The debonding of the epoxy coating can readily be seen along the entire length of this specimen.

Figure 1 - Photograph of a typical joint reinforcing dowel removed from a concrete pavement slab after only a few years of service.

Pyć et al. (2000) suggests that epoxy coated reinforcement does not provide any additional service life for concrete bridge decks and recommends that the Virginia Department of Transportation discontinue the use of epoxy coated reinforcement in bridges. There is considerable disagreement in the current literature about the effectiveness of epoxy coated reinforcement. In "ideal" laboratory settings, epoxy coating tends to prevent the oxidation of reinforcing steel. However, for practical real-world use the coating does not appear to be effective.

In contrast to epoxy coated reinforcement, stainless steel reinforcement will not oxidize regardless of any scratches or dings during handling and installation. Unfortunately, stainless steel reinforcement is expensive. To reduce the expense and still maintain the corrosion resistant advantages, stainless steel-clad reinforcing bars are a viable option. These bars have been shown to resist corrosion virtually as well as solid stainless steel reinforcement bars but cost about 50% less (Clemena, 2002; Better, 2001; Yunovich and Thompson, 2003).

Galvanized reinforcing steel has been used in concrete structures since the 1930s. Hot dip galvanizing is a process of immersing black steel in molten zinc at about 450°C (842°F) which creates a metallurgically bonded coating of zinc and zinc-iron alloys on the base steel (Yeomans, 2002). The zinc coating has a higher oxidation potential then steel and it readily gives up electrons during oxidation (Better, 2001). Studies have shown that galvanized reinforcement resists chloride levels of about 2.5 times higher than black steel. These studies also indicate that galvanized coatings may delay the time to the onset of corrosion of the underlying steel by up to 5 times (Yeomans, 2002).

The MMFX Steel Corporation was established in 1998 and claims that MMFX steel offers corrosion resistance properties similar to stainless steel with superior strength and mechanical properties (MMFX, 2002). Research has shown that MMFX steel reinforcement exhibits a higher strength than typical Grade 60 reinforcement. Due to this higher strength, and to some extent the lack of a distinct yield point, caution must be exercised when specifying MMFX steel reinforcement (Ansley, 2003). Ansley (2003) suggests that a blind substitution of MMFX reinforcement for typical Grade 60 reinforcement should be avoided. MMFX steel technology is still relatively new and more performance data is needed to verify long term corrosion resistance of these bars.

5. DOCUMENT REVIEW

Sources used to obtain information specifically relative to the I-15 Reconstruction Project include:

- UDOT specifications;
- Previous UDOT inspection reports;
- Construction drawings;
- Field reports and documents during construction;

5.1 UDOT Specifications

UDOT specification 506, “Concrete Structures,” was applicable during the I-15 Reconstruction Project. The ninth revision of this specification dated September 7, 1999 was made available for review for this study. This specification allows the use of fogging during concrete deck placement. However, only fogging equipment that incorporates “the use of compressed air misters that atomize the water, producing a very fine mist and not a spray,” was allowed. The specification also states that the “misters shall not be aimed in a direction lower than horizontal at 1.5 m (4.9 ft) above the concrete surface.”

Specification 506 also calls for a full seven days of wet curing on all bridge decks and approach slabs. According to the specification, an approved curing compound should be applied to the slab such that no portion of the concrete is exposed to the drying effects of the atmosphere for more than 20 minutes. As soon as the concrete can support it, the concrete should be covered with a moisture retaining material (i.e., cotton or burlap mats) and a series of soaker hoses to continually provide curing water to the concrete surface. With the exception of the curing compound, the specification appears to be in accordance with the current recommended practices for curing silica fume concrete. As stated above, Morin et al. (2002) recommends eliminating the use of curing compounds for high performance concrete (HPC) because the compounds "will prevent the penetration of curing water that is needed to control the development of autogenous shrinkage."

5.2 UDOT Investigation Spreadsheets

UDOT field personnel made observations of the new I-15 bridges during the final year of the I-15 Reconstruction Project. The results of these observations are contained in two separate spreadsheets entitled "I-15 Reconstruction Bridge Deck Concerns Summary" and "Preventive Needs for I-15 Decks."

For the "I-15 Reconstruction Bridge Deck Concerns Summary" spreadsheet, some bridge decks were inspected on the top side and others were inspected from below. These inspections were intended to identify which bridges UDOT should be most concerned with. Longitudinal, transverse, and diagonal cracks were counted separately and a severity ranking of high, medium, or low was assigned to each deck. The version of this spreadsheet that was made available for this study, dated January 3, 2001, was analyzed and included in the database for all 142 bridges.

The inspections performed for the "Preventive Needs for I-15 Decks" spreadsheet were all conducted on the topside of the bridge deck. This inspection process did not include all of the new bridges of I-15. Sixty-six of the seventy-one bridges studied for this report were included in the "Preventive Needs for I-15 Decks" spreadsheet. These inspections took place between March and May of 2001. To facilitate the inspection process, UDOT established a "Priority Ranking" scale of 1 to 5 to rank the amount of visible bridge deck cracking. In this scale, decks ranked 5 are the best and decks ranked 1 are the worst. This is opposite to the Cracking Severity Index scale (see Section 6.4) that was established for this study. In order to make worthwhile comparisons between the two data sets, the UDOT priority ranking numbers were revised to show high priority bridges with a 5 and low priority bridges with a 1. This adjusted data was then entered into the database.

It should be noted that these two separate inspections, the top side by UDOT and the underside for this study, were done by different individuals and the rankings are subjectively based upon visual inspections.

5.3 I-15 Bridge Construction Drawings

As part of this study, the construction drawings for all 71 studied bridges were reviewed. This entailed examination of approximately 1000 sheets of structural drawings. Data extracted from these drawings and inserted into the database include:

- Girder depth;
- Girder spacing;
- Deck thickness;
- Deck concrete design f'c;
- Abutment type;

- Wingwall condition;
- Approach slab dowel condition;
- Construction details and comments

5.4 I-15 Construction Documents and Field Reports

A limited number of Field Design Change (FDC) Memos were acquired and reviewed. The majority of available FDCs dealt with changing the specified deck placement procedures. The contractor often requested that the deck be placed in a single operation rather than staged sequencing. Other FDCs requested longitudinal construction joints in the deck to accommodate the tight quarters caused by a neighboring bridge. For Bridge No. 216 the contractor requested that the deck be constructed without precast deck panels. Given the limited available information, it appears that all of the requested changes were authorized.

Two of the available FDCs addressed some minor cracking in the ends of pre-stressed concrete girders. This cracking is thought to have been a function of inadequate pre-stressing details. There is no indication of any ongoing problems, however further investigation is beyond the scope of this study.

Scores of quality control documents that were generated during the construction phase of the I-15 Reconstruction Project were also reviewed. These documents included daily inspection reports, concrete placement reports, and concrete cylinder break information pertaining to the bridge decks. This data is included in the database in six different tables. Each table represents a section of deck placed. For instance, if an entire deck was placed at one time, the bridge will only have a data set in the "1st Placement Sequence" table. The deck of Bridge No. 174 was placed in six separate operations and consequently has data sets in all six placement tables.

6. FIELD OBSERVATIONS

All 71 bridges listed in Table 1 were visually inspected on numerous occasions. These inspections were conducted between December 2002 and August 2003. Every bridge was observed at least twice.

6.1 Underside Inspections

Comprehensive visual inspections of the underside of all 71 bridges were conducted. The majority of these inspections took place during rain or snow events to permit observation of water on the underside of the cracked bridge decks. All of the bridge undersides were observed on more than one occasion. Water was observed on the deck undersides of 70% (50 of the 71 bridges) of the bridges. This water was generally noticeable at cracks. Even without the presence of water, existing cracks could regularly be seen on the deck underside due to the presence of efflorescence.

Efflorescence is a crystalline deposit of soluble salts on the surface of concrete. When these salts come into contact with carbon dioxide, crusty, white deposits are formed. The presence of efflorescence is not necessarily a sign of deteriorating concrete; the deposits often originate from salt placed on the bridge decks in the winter to control ice buildup (Neville, Aug. 2002). Regardless of whether the efflorescence salts were placed on the deck or stripped from the concrete, the efflorescence laced cracks are evidence that water is migrating through the bridge deck.

6.2 Topside Inspections

In addition to the comprehensive underside inspections, the topside of eight bridge decks between 4500 south and 1000 south were observed in August of 2003. This was possible because UDOT crews had some I-15 lanes closed. All of the observed decks had large numbers of visible surface cracks on the topside. Some of these cracks had been routed and epoxied but the majority had not been treated. Most of the cracks appeared to be early age, plastic shrinkage, surface cracking, as they were not all visible from below.

6.3 Photographs

Bridge inspections were documented with numerous digital photographs. Some of these photographs are shown in Appendix A. All of the digital photograph files have been turned over to UDOT to be combined with the Department's current working bridge data inventory. These photos will serve as a comparison reference point for the bridge decks as they age.

6.4 0 to 5 Cracking Severity Ranking Scale

In order to make useful comparisons, all of the bridge decks were ranked with a Cracking Severity Index Number (CSIN). These ranks were assigned according to the information gathered during the underside inspections. The CSIN scale was developed for this study in order to provide a measure of the relative level of cracking that is visible from the underside of each bridge deck. Due to the relatively young age of the bridges studied, none of the CSIN rank definitions account for cracking caused by corrosion of reinforcing steel. The 0 to 5 CSIN ranks are defined below.

0 – No visible cracking on the deck underside.

1 – Minor cracking on the deck underside. These decks generally have less than ten cracks which are all relatively minor.

2 – Moderate cracking on the deck underside generally confined to the outer quarter of the bridge spans. These decks have moderate amounts of diagonal cracks that are at least 12 inches in length. These decks may also have limited transverse cracks. However, cracks are not generally visible on the underside of the deck within the center half of the bridge spans.

3 – Moderate cracking on the deck underside over the entire length of the bridge. These decks generally have regularly spaced transverse cracks (approximately 10 feet on center) over the entire bridge deck as seen from the underside. These decks may also have some diagonal cracks which are generally at the bridge span ends.

4 – Heavy cracking on the deck underside generally confined to the outer quarter of the bridge spans. These decks have many diagonal cracks occurring within the outer quarter of the spans, and may also have transverse cracks. However, cracks are not generally visible on the underside of the deck within the center half of the bridge spans.

5 – Heavy cracking on the deck underside over the entire length of the bridge. These decks generally have closely spaced transverse cracks (approximately 5 feet on center or less) over the entire length of the bridge deck as seen from the underside. These decks may also have diagonal cracks which are generally at the bridge span ends.

Table 3 lists the 71 studied bridges and their corresponding CSIN rankings. Figure 2 illustrates the typical types and amount of cracking associated with each CSIN rank.

Table 3 - The CSIN rankings associated with all 71 bridges studied.

RFP Bridge Number	CSIN	RFP Bridge Number	CSIN	RFP Bridge Number	CSIN	RFP Bridge Number	CSIN
3	1	52	3	145	3	180	3
3.5	0	54	1	147	3	182	1
8	1	56	1	148	1	196	5
10	1	60	3	149	1	198	5
12	1	62	2	150	2	200	3
14	1	64	1	151	3	202	5
16	2	66	1	152	1	212	3
18	1	70	5	153	1	214	3
20	1	72	5	154	2	216	5
22	1	74	1	155	1	218	5
23	5	76	1	156	1	220	3
26	5	112	5	158	2	222	3
27	3	114	3	160	2	224	5
28	5	116	3	162	2	226	5
29	1	138	1	168	3	230	5
30	1	140	2	170	3	702	5
32	1	142	1	174	3	12002	1
50	3	144	2	176	3		

(a) – Typical CSIN = 0

(b) – Typical CSIN = 1

(c) – Typical CSIN = 2

(d) – Typical CSIN = 3

(e) – Typical CSIN = 4

(f) – Typical CSIN = 5

Figure 2 - Photos of the undersides of new I-15 bridge decks depicting typical amounts of cracking associated with each CSIN value. The "lines" observed on the deck in (a) are precast panel joints and not cracks.

It is important to note that only one bridge in the study (Bridge No. 3.5) was classified with a CSIN of 0. Every other bridge had more than one visible crack on the deck underside. No bridges were ranked with a CSIN of 4. Heavy cracking on any bridge deck was not limited to the bridge span ends.

6.5 Evidence of Moisture

The majority of underside inspections were done during and shortly after rain or snow events to determine whether or not the visible cracks were allowing water to permeate through the bridge decks. This proved worthwhile since most bridges, 50 of the 71 studied (70%), were observed with moisture present on the underside of the bridge deck on at least one occasion. Two bridge decks (Bridge No. 52 and Bridge No. 224) were observed with water actually dripping from cracks on the deck underside.

Another important observation was that 26 of the 71 (37%) bridge structures had water leaking through the face of the concrete abutment walls. This has long term implications and the information was therefore entered into the database.

Two bridges were noted with moisture on the underside of the decks many days after any rain or snow. Bridge No. 180, a PC bridge constructed with precast deck panels, was observed with moisture present at some of the precast panel joints on the deck underside on January 4, 2003. This was roughly four days after any rain or snow had fallen. Also, the bridge topside surface was dry at this time. Similarly, Bridge No. 72, a PC bridge constructed without precast deck panels, was observed on July 8, 2003 with moisture at some of the deck cracks on the underside of the bridge. This observation was made many days after any rain had fallen and indicates that some type of internal reservoir is holding water.

6.6 Non I-15 Reconstruction Project Bridges

A brief survey of the underside of over 50 bridges beyond the limits of the I-15 Reconstruction Project was conducted. A listing of these bridges can be found in Appendix C. These bridges range in age from 1 to 40 years and are located along the Wasatch Front from Lehi to Farmington. A ranking method similar to the 0 to 5 CSIN scale was used to quantify the amount of visible cracking on the underside of the bridges. It was observed that there were projects with cracking equivalent to all CSIN rankings. Many of the bridges have cracks equivalent to CSIN 5. However, there are also many bridges that appear to have far less cracking then the average amount found on the I-15 Reconstruction Project bridges.

It should be noted that there were examples of concrete girder and steel girder bridges with limited amounts of cracking. This can only be attributed to design and construction procedures that were implemented to minimize shrinkage cracking.

7. I-15 BRIDGE DATABASE

A database was constructed using Microsoft Access Software. This database contains some information pertaining to all 142 of the new I-15 bridges. The data that was acquired directly from UDOT spreadsheets was entered into the database for all of the bridges. Data that was collected by other means was only acquired for the 71 bridges listed in Table 1. Table 4 lists 57 variables in the database by which the data may be sorted for the 71 studied bridges. Item 25 (CSIN), was used as the dominate variable for bridge deck comparison in these investigations.

Table 4 - List of database variables by which the data may be sorted. The entries with (UDOT) come from the UDOT observation spreadsheets.

1. RFP Bridge Number	30. Skew Angle at Bridge Ends (deg.)
2. UDOT Inventory Number	31. Precast Concrete Deck Panels (Y/N)
3. Part of Research Study	32. Number of Placements
4. Location of Bridge	33. Volume of Concrete Placed (m^3)
5. Type of Structure	34. Weather During Placement
6. Number of Spans	35. Avg. Atmospheric Temp During Placement (°C)
7. Longest Span (m)	36. Weekday of Placement
8. Bridge Length (m)	37. Number of 7-Day Cylinders
9. Average Bridge Width (m)	38. Avg. 7-Day Cylinder Strength (kPa)
10. Maximum Structural Depth (m)	39. Number of 14-Day Cylinders
11. Girder Type	40. Avg. 14-Day Cylinder Strength (kPa)
12. Maximum Girder Depth (m)	41. Number of 28-Day Cylinders
13. Maximum Girder Spacing (m)	42. Avg. 28-Day Cylinder Strength (kPa)
14. Deck Design f'c (kPa)	43. Number of Days After Deck Placement That a Curing Check was Reported (days)
15. Deck Thickness (m)	44. Cracking Density (UDOT)
16. Abutment Type	45. Equivalent Transverse Crack Spacing (UDOT)
17. Wingwall Condition	46. Equivalent Cracked Area (%) (UDOT)
18. Number of Fin Walls	47. Inadequate Rebar Cover (UDOT)
19. Fin Wall Length (m)	48. Inadequate Deck Thickness (UDOT)
20. Fin Wall Height (m)	49. Concern for Deck Condition (UDOT)
21. Approach Slab Doweled to Bridge Deck (Y/N)	50. Number of Diagonal Cracks (UDOT)
22. Transverse Cracking on Deck Underside (Y/N)	51. Severity of Diagonal Cracks (UDOT)
23. Longitudinal Cracking on Deck Underside (Y/N)	52. Number of Longitudinal Cracks (UDOT)
24. Diagonal Cracking on Deck Underside (Y/N)	53. Severity of Longitudinal Cracks (UDOT)
25. Cracking Severity Index Number	54. Number of Transverse Cracks (UDOT)
26. Moisture Observed Below Deck (Y/N)	55. Severity of Transverse Cracks (UDOT)
27. Leaking Abutment Observed (Y/N)	56. Scheduled Deck Treatment (UDOT)
28. Deck Pour Date	57. Revised Priority for Deck Treatment (UDOT)
29. Deck Area (m^2)	

Literally hundreds of queries were conducted in an attempt to isolate specific variables that may be responsible for the various crack types that exist in the new I-15 bridge decks. The different types of observed cracking were categorized as transverse, diagonal, and longitudinal. Transverse cracks are perpendicular to the flow of traffic and longitudinal cracks are parallel to traffic flow. For bridges constructed with precast concrete deck panels, if water was observed at a panel joint then the joint was considered to be a transverse crack. A diagonal crack is any crack that does not fit the longitudinal or transverse cracking descriptions. The results of the database queries are given in Section 8.

8. DATABASE QUERY RESULTS

The database was queried hundreds of different ways. Many of the attempted queries did not reveal any significant trends. Other queries provided strong data to support the observations of other researchers and led to conclusions and recommendations. A sample set of database queries is given in Appendix B.

8.1 Silica Fume Concrete

All of the bridge decks constructed during the I-15 Reconstruction Project contained silica fume concrete. The specifications for the cast-in-place portions of the decks called for a silica fume content of 5% by weight of cementitious materials. According to comments in the UDOT "Preventive Needs for I-15 Decks" spreadsheet (see Section 5.2), top surface cracks were observed on 62 of the 66 bridges (94%). For the other four bridges, two were not yet open to traffic, one had a copolymer and broadcast aggregate overlay system applied to the deck topside, and there were no comments pertaining to the other bridge. The spreadsheet also indicated that 21 of the 66 bridges (32%) had "extensive" amounts of cracking. The bridge deck topsides that were observed for this study all had significant amounts of visible surface cracks (see Section 6.2).

If the silica fume concrete was improperly cured, then it could be assumed that this contributed to the shrinkage cracking. However, if the decks were properly cured, then the silica fume did not likely contribute to the shrinkage cracking (Whiting and Detwiler, 1998; Lafave et al., 2002; Hadidi and Saadeghvaziri, 2003). The set of construction documents and daily field reports that were reviewed for this study were incomplete for many of the bridges. For the available documents, water curing was only mentioned to have taken place on 51% of the bridge decks and reports of water curing taking place for the entire specified seven days were only available for 14% of the bridge decks. However, this issue is probably not accurately represented because of the limited amount of written documentation that was available for review.

8.2 Cracking Type and Distribution

The types of cracking and number of bridges associated with each CSIN ranking is given in Table 5. This table provides a summary of the various combinations of cracking type and the extent of bridge deck cracking found in this study.

Visible cracks are present on the deck underside of all but 1 of the 71 studied bridges. Diagonal cracking is found on 62 bridges (87%) and is generally an indication of transverse restraint. Nearly all of the diagonal cracking was observed near the bridge abutments or interior bents. Transverse cracking is also common and can be seen on 47 bridges (67%). The transverse cracks appear to have been caused by concrete shrinkage. Many bridge decks have transverse cracks at regularly spaced intervals across the span. Only 11 bridges (15%) have visible longitudinal cracks on the deck underside. These longitudinal cracks are quite limited and of short length. All bridges with longitudinal cracking also have diagonal cracking and all but three have transverse cracking. Transverse and diagonal cracking is found together on 39 of the 71 bridges (55%).

Table 5 - The number of bridges and type of cracking associated with each CSIN rank.

CSIN	Total Number of Bridges	Number of Bridges Corresponding to Cracking Type						
		D Only	T Only	L Only	D and T	D and L	T and L	D and T and L
0	1	0	0	0	0	0	0	0
1	27	18	3	0	4	2	0	0
2	9	2	0	0	5	1	0	1
3	19	0	3	0	10	0	0	6
4	0	0	0	0	0	0	0	0
5	15	0	2	0	12	0	0	1
Totals	71	20	8	0	31	3	0	8

Notes: CSIN = Cracking Severity Index Number
D = Diagonal Cracks
T = Transverse Cracks (Perpendicular to Traffic Flow)
L = Longitudinal Cracks (Parallel to Traffic Flow)

8.3 Date of Construction

For the bridges studied, deck placement began in April 1998 and concluded with Bridge No. 198 in March 2001. As shown in Figure 3, the deck cracking tends to be greater for bridges that were constructed later in the project. This is particularly true for the concrete girder bridges. The steel girder bridges have inherently more cracking so that the trend is not as noticeable.

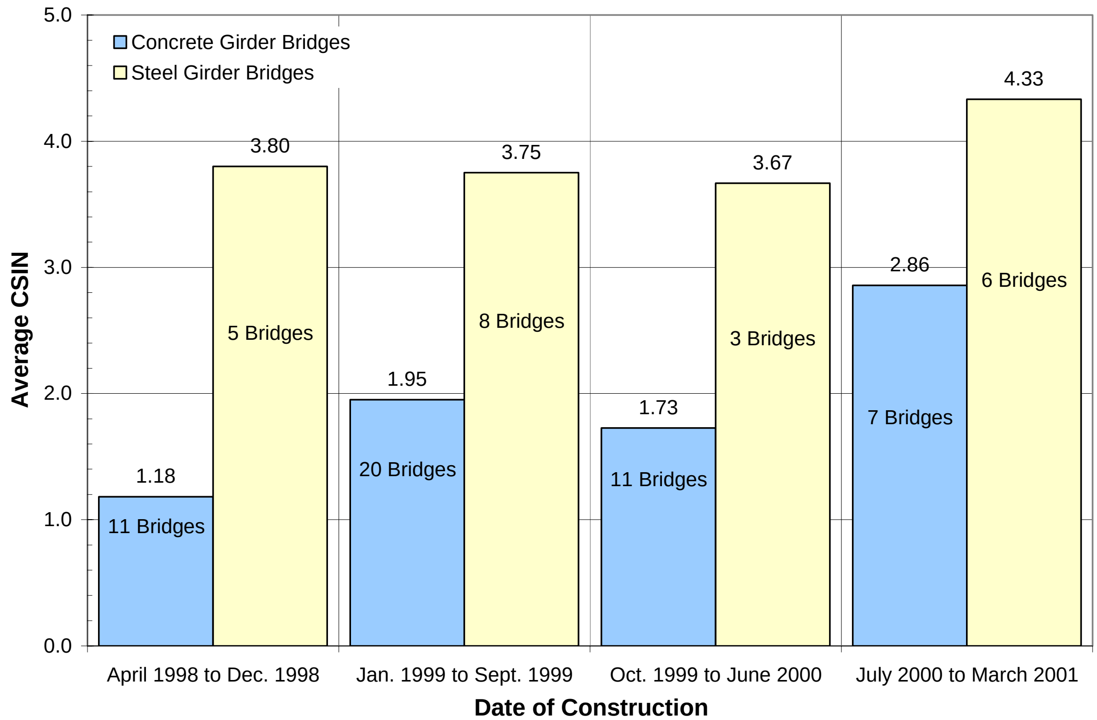
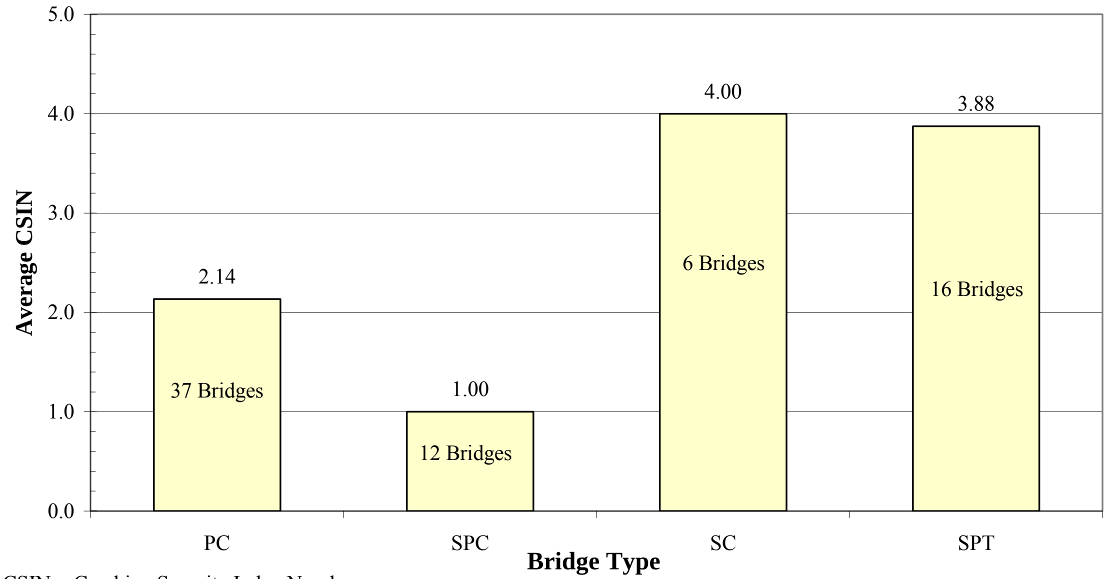


Figure 3 - CSIN vs. date of construction for the studied bridges.

Figure 3 indicates that some Quality Control (QC) issues may have been present during the I-15 Reconstruction Project, particularly towards the end.

8.4 Girder Type

All 71 of the studied bridges were initially compared. However, it quickly became apparent that different mechanisms were contributing to the cracking of the different bridge types. The amount of cracking on steel girder bridges and concrete girder bridges was typically very different. As shown in Figure 4, the average severity of cracking observed on the steel girder bridges is considerably higher than cracking found on the concrete girder bridges. Figure 5 is a histogram of the CSIN rankings for the different bridge types. All but one of the steel girder bridges received CSIN rankings of 3 or greater. The SC bridges have an average CSIN value of 4.00 and the average CSIN for the SPT bridges was 3.88. In contrast, the average CSIN values for the PC and SPC bridges were 2.14 and 1.00 respectively.



CSIN = Cracking Severity Index Number
PC = Pre-Stressed Concrete Girders with Reinforced Deck
SPC = Spliced Post-Tensioned Concrete Girders with Reinforced Deck
SC = Steel Girders with Reinforced Deck
SPT = Steel Girders with Transversely Post-Tensioned Deck

Figure 4 - Comparison of the average CSIN values for the different bridge types studied.

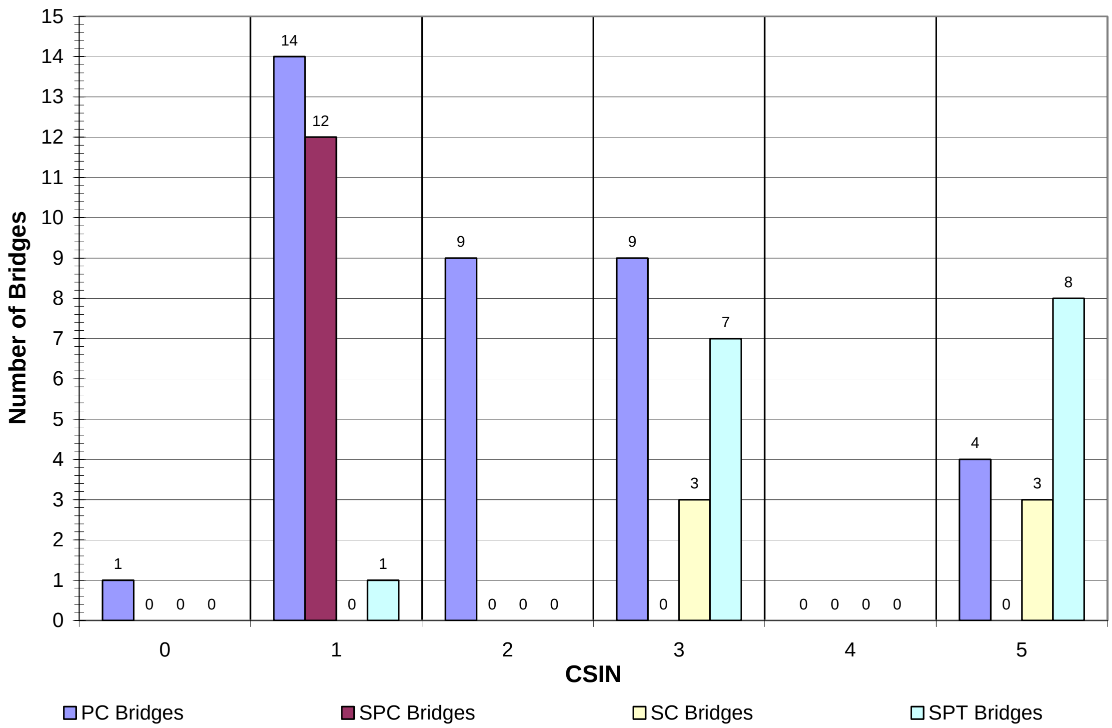


Figure 5 - Histogram of the number of each bridge type associated with each CSIN rank.

The steel girder bridges have such high CSIN values relative to the concrete girder bridges that a meaningful study comparing all 71 bridges was fruitless. Consequently, the steel girder bridges and the concrete girder bridges were generally analyzed separately. For some queries, these sub-sets were broken down even further into the different types of concrete and steel girder bridges.

8.4.1 Steel Girder Bridges

The steel girder bridge deck cracking is dominated by transverse cracks. All decks on steel girders exhibit transverse cracking and 82% also have diagonal cracking. Bridge No. 50, an SC bridge, is the only steel girder bridge exhibiting longitudinal cracks. This is partially due to the transverse post-tensioning in the decks of the SPT bridges which tends to close longitudinal cracks.

There are numerous variables contributing to the large amount of cracking found on the steel girder bridges. The steel girder bridges are generally longer than the concrete girder bridges. The average steel girder bridge length is 123.3 m (404.5 ft) while the average concrete girder bridge length is only 57.5 m (188.6 ft). Also, 10 of the 22 (45%) steel girder bridge decks were placed in a single operation. This includes Bridge No. 230 (the longest structure studied) which is 246 m (807 ft) long.

The coefficients of thermal expansion are 6.5×10^{-6}/°F and 5.5×10^{-6}/°F for steel and concrete respectively (MacGregor, 1997). This creates an 18% thermal expansion differential between the concrete deck and the steel superstructure. This difference will put the concrete deck into compression as the temperature drops below the construction temperature. However, as the temperature rises above the construction temperature the deck is put into tension by the expanding steel girders. When the thermal tensile stress is combined with the internal stresses from concrete shrinkage, the tensile capacity of the concrete may be exceeded and cracks will form.

8.4.1.1 Transverse Post-Tensioning

Transverse post-tensioning in bridge decks allows the girder spacing to increase because of the precompression and uplifting action of the post-tensioning strand. Also, the post-tensioning force will close up any longitudinal cracks. As expected, none of the SPT bridges have visible longitudinal cracking on the deck slab underside. However, all of the SPT bridges have transverse cracking and all but two have diagonal cracking. These diagonal cracks are generally present near bents and abutments.

The transverse post-tensioning present in the decks of the SPT bridges may well contribute to the diagonal cracking that is present near the abutments and bents. The concrete deck will creep under the applied transverse post-tensioning loads. This creep is restrained where the deck is connected to the diaphragms and abutments. The strain differentials that develop from this restraint cause tensile stresses in the deck which manifest themselves as cracks if the tensile capacity of the concrete is exceeded.

8.4.2 Concrete Girder Bridges

There are two types of concrete girder bridges within this study. PC bridges were built with pre-stressed, simply supported concrete girders. SPC bridges were built with pre-stressed concrete girder segments that were then post-tensioned together after the deck was constructed.

8.4.2.1 SPC Bridges

This study contains twelve SPC bridges. As seen in Figure 4, these SPC bridges all received CSIN rankings of 1. The construction details of these bridges are essentially the same. These bridges exhibit a consistently high quality and have low CSIN rankings.

SPC bridge decks perform well because of their configuration. The girders of SPC bridges were longitudinally post-tensioned after the deck concrete reached the design compressive strength. This longitudinal post-tensioning in the girders compresses the deck and closes up any transverse shrinkage cracks that may have formed during the curing stage. Additionally, all of the SPC bridges are long, single span structures. This single span design creates a large positive moment in the deck and girder system. This positive moment puts the deck into compression which also helps to close up any cracks that may be present in the concrete bridge deck. SPC bridge girders were also shored at third points during the deck construction. When this shoring is removed, the deck is compressed even further. Since there are no permanent interior supports for the SPC bridges, there are no negative moment regions to put the deck concrete into tension.

Any post-tensioning loss that may occur due to creep will not adversely affect the deck because the losses are counteracted by additional compressive stresses in the deck caused by increased deflection. Lounis and Mirza (1997) found that “the service load performance of this system is much better than that of the conventional continuous precast I-girder system.”

All studied SPC bridges have minor diagonal cracking near the abutment corners. Some SPC bridges also have minor diagonal cracking where the girders meet the abutments. There were no transverse or longitudinal cracks observed on the SPC bridges. The observed diagonal cracking may be a result of the abutment restraint and shear lag due to the post-tensioning of the girders. This cracking is relatively minor because of the use of closure pourback strips to accommodate the post-tensioning of the girders after the deck placement. These pourback strips alleviate some of the restraint that is associated with semi-integral abutments

Five SPC bridges (42%) were observed with moisture present on the deck underside. This moisture was very minimal and only present at some of the diagonal cracks near the abutment. Moisture at precast deck panel joints was never observed on an SPC bridge. This is due to the longitudinal post-tensioning forces and large positive moment that compress the deck together and prohibit any water from entering. Five SPC bridges were also observed with water leaking through the concrete abutment walls

For the studied bridge decks, 47 of the 71 (66%) have transverse cracks. If the set is broken into bridges with and without longitudinal post-tensioning, 47 of 59 (80%) non-SPC bridge decks have transverse cracks while none of the SPC bridges exhibit transverse cracking. As mentioned above, this is mainly due to the longitudinal post-tensioning forces applied to the SPC bridges.

If care is taken in the deck design and adequate concrete cover is provided for the post-tensioning tendons, longitudinal post-tensioning of bridge decks could alleviate the transverse through-cracks. Generally, in order to accomplish longitudinal post-tensioning in the deck, the girder capacity would have to be increased in order to offset the downward loading effect of the post-tensioning which would act in addition to any dead and live loads applied to the deck. Longitudinal post-tensioning of bridge decks would also require that restraint issues at the abutments be addressed.

Longitudinal post-tensioning in a concrete bridge deck could possibly be accomplished by incorporating a t-beam analysis into the design. The deck would act compositely with the girder to form the t-beam section. If the longitudinal post-tensioning was placed at the centroid of the composite t-beam section, then the downward loading effects of the longitudinal post-tensioning could be minimized.

8.4.2.2 PC Bridges

The results of this study show more variability in the CSIN rankings for PC bridges than for the other bridge types. Only four concrete girder bridges (70, 72, 224, and 226) received CSIN rankings of 5. These four are all PC bridges and were constructed without precast concrete deck panels. As a result of the severe cracking of this subset of PC bridges, these 4 bridges were generally analyzed independently of the remaining 33 PC bridges.

Water was observed at a number of precast panel joints on the underside of several PC bridges, indicating that the upper cast-in-place slab is cracked. This is to be expected due to the "notch effect" and corresponding stress concentrations in the composite deck (Abendroth, 1995).

8.5 Abutment Type

There are three types of abutments evaluated in this study.

- Integral Abutments – an integral abutment is designed such that the girders, end diaphragm, bridge deck, abutment, and approach slab are all rigidly connected.
- Semi-Integral Abutments – a semi-integral abutment is similar to an integral abutment except that the abutment is not rigidly attached to the other elements, i.e. approach slab, bridge deck, end diaphragm, and girders.

- Expansion Abutments – an expansion abutment is one where the bridge deck, end diaphragm, and girders are separated from the abutment and approach slab by means of a mechanical expansion joint and bearings.

Table 6 shows the types of abutments associated with each bridge type. For steel girder bridges, 55% have expansion abutments, 9% have semi-integral abutments, and 36% have integral abutments. For concrete girder bridges, 71% of have integral abutments and 29% have semi-integral abutments. There are no concrete girder bridges in the study with expansion joints. Due to the large amount of cracking present on the steel girder bridges and the limited distribution of abutment types present on the PC bridges, it is difficult to draw any conclusions relating abutment type and cracking severity. However, it is interesting to note that all steel girder bridges with integral abutments have diagonal cracking present on the bridge deck undersides near restraining elements (abutments, bents, and diaphragms). Even though the expansion joints alleviate most of the longitudinal restraint between the bridge deck and abutment, the diaphragms still provide enough restraint to promote cracking, particularly in the transversely post-tensioned decks.

For concrete girder bridges with integral abutments, 89% have diagonal cracking near the abutments. This is because of the large amount of restraint that integral abutments provide. Also, all nine of the studied bridges that received CSIN rankings of 2 are PC bridges with integral abutments. The use of pourback strips will help to alleviate these cracks by allowing most of the deck concrete to shrink and set before any rigid attachment is made to the abutments.

Table 6 - The types of abutments associated with each bridge type.

Abutment Type	SPT Bridges	SC Bridges	PC Bridges	SPC Bridges
Integral	6	2	35	0
Semi-Integral	2	0	2	12
Expansion	8	4	0	0

8.6 Volume of Concrete Placed

Normal concrete shrinks as it cures. This shrinkage creates tensile forces in restrained conditions. The amount of shrinkage increases with an increase in the volume of concrete placed. This is evident by the increased amount of cracking found in bridge decks that were constructed with large volumes of concrete placed at one time. Figure 6 illustrates the relationship between the volume of bridge deck concrete placed in a single operation and the severity of cracking on the bridge deck. The relationship is shown separately for concrete and steel girder bridges. The concrete girder bridges were first sorted by original deck placement size and then divided into quarters. The CSIN and volume were averaged for each quarter group of bridges. These average values were then plotted. The same procedure was used to display the data for the steel girder bridges. Trendlines were fitted to each set of data using a least squares analysis. The trend for both concrete and steel girder bridges indicates that as larger volumes of concrete are placed in a single operation, the bridge decks will generally experience more cracking.

Cracking is the only way for concrete to relieve itself of the internal stresses that develop during shrinkage in restrained conditions. Rigid attachment to abutments, bents, and diaphragms will restrain the deck concrete as it shrinks. Composite action between the bridge deck and girders may also provide the restraint necessary for cracking to occur. Bridges constructed with monolithically cast concrete girders and bridge decks were observed during the non I-15 bridge survey (see Section 6.6). These bridges typically had small amounts of cracking on the deck undersides. For concrete bridge decks that are to be rigidly attached to stiffer abutments, bents, and diaphragms, the proper use of pourback strips will alleviate some restraint.

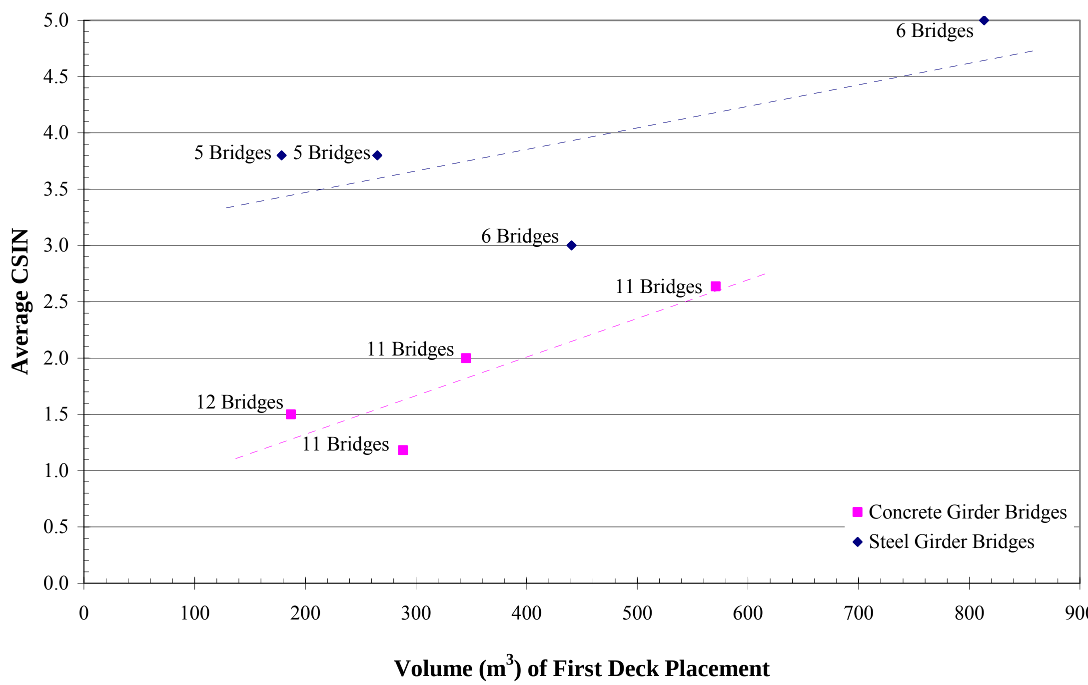


Figure 6 - CSIN vs. the volume of concrete that was placed during the initial deck placement. The trendlines were created using a least squares analysis.

8.7 Bridge Deck Length

The length of a bridge will contribute to the amount of cracking. Figure 7 shows the relationship between bridge length and CSIN for each of the four different bridge types. Each data set was fit with a trendline using a least squares analysis. For most bridge types, the data indicates an increase in cracking severity for longer bridges. There is no apparent trend for SPC bridges because they all have the same CSIN ranking. An important item to note is the different average bridge length for the various bridge types. On average, steel girder bridges are considerably longer than concrete girder bridges.

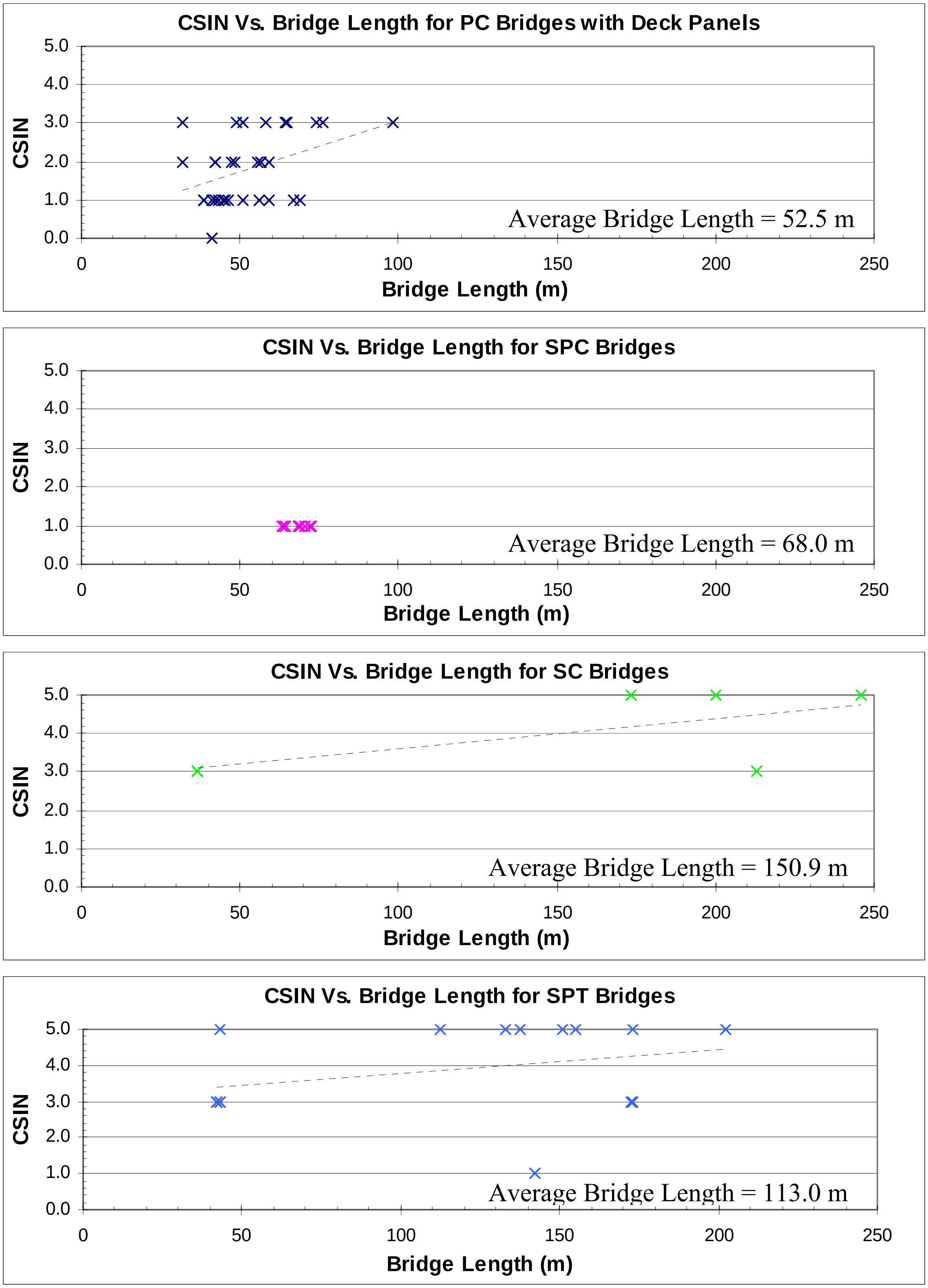


Figure 7 - CSIN vs. bridge length for the four different bridge types. The trendlines were created using a least squares analysis.

8.8 Bridge Deck Width

Figure 8 shows a series of plots of CSIN vs. average bridge width for each of the four different bridge types. Each data set was fit with a trendline using a least squares analysis. The trend for increased cracking as a function of bridge width is not as obvious as the trend for cracking as a function of length. Relative to the length of a bridge, the width is generally quite small. This small dimension limits the amount of shrinkage that the concrete will see in this direction. This is why so much transverse cracking is present in the study and relatively small amounts of longitudinal cracking. The plots of SC bridges and SPC bridges show less data points than bridges. This is because some of the bridges have the same width and CSIN score and the data points overlay each other.

For steel girder bridges, the trendlines indicate that a bridge will crack less as the deck gets wider. This is an illusion. The deck cracking on steel girder bridges has generally resulted from long uncontrolled concrete placements that are restrained at abutments and bents. The CSIN ranking scale as defined in Section 6.4 does not directly differentiate between longitudinal and transverse cracking. CSIN values are typically controlled by length-induced cracking rather than width-induced cracking because of the larger length dimension. For SC bridges, the length of a bridge tends to decrease as the width increases and previous queries show that CSIN increases with length. This is why the CSIN vs. width trend has a negative slope for the SC bridges of Figure 8. The cracking on the SPT bridges is also dominated by long, restrained concrete placements. Additionally, regardless of the width of the bridge, the transverse post-tensioning tends to exacerbate cracking near restraining elements such as abutments, bents, and diaphragms.

Only eleven of the studied bridges have visible longitudinal cracking on the underside of the deck. All recorded longitudinal cracking was minor relative to the transverse and diagonal cracking that was observed. Figure 10 shows the relationship between average bridge width and CSIN ranking for the bridges with longitudinal cracks. Most of these bridges are PC bridges with precast concrete panels. The only exceptions are Bridge No. 226 and Bridge No. 50. Bridge No. 226 is a PC bridge that was constructed without precast panels. This bridge was eliminated from many of the database queries due to the extreme amount of cracking that is present. Please refer to Section 8.9 for more information about Bridge No. 226.

Bridge No. 50 is a short, single-span, SC bridge with integral abutments. This bridge has many transverse and diagonal cracks along with the longitudinal cracks shown in Figure 9. Figure 10 illustrates that for bridges with longitudinal cracking, cracking severity increases as bridge decks get wider.

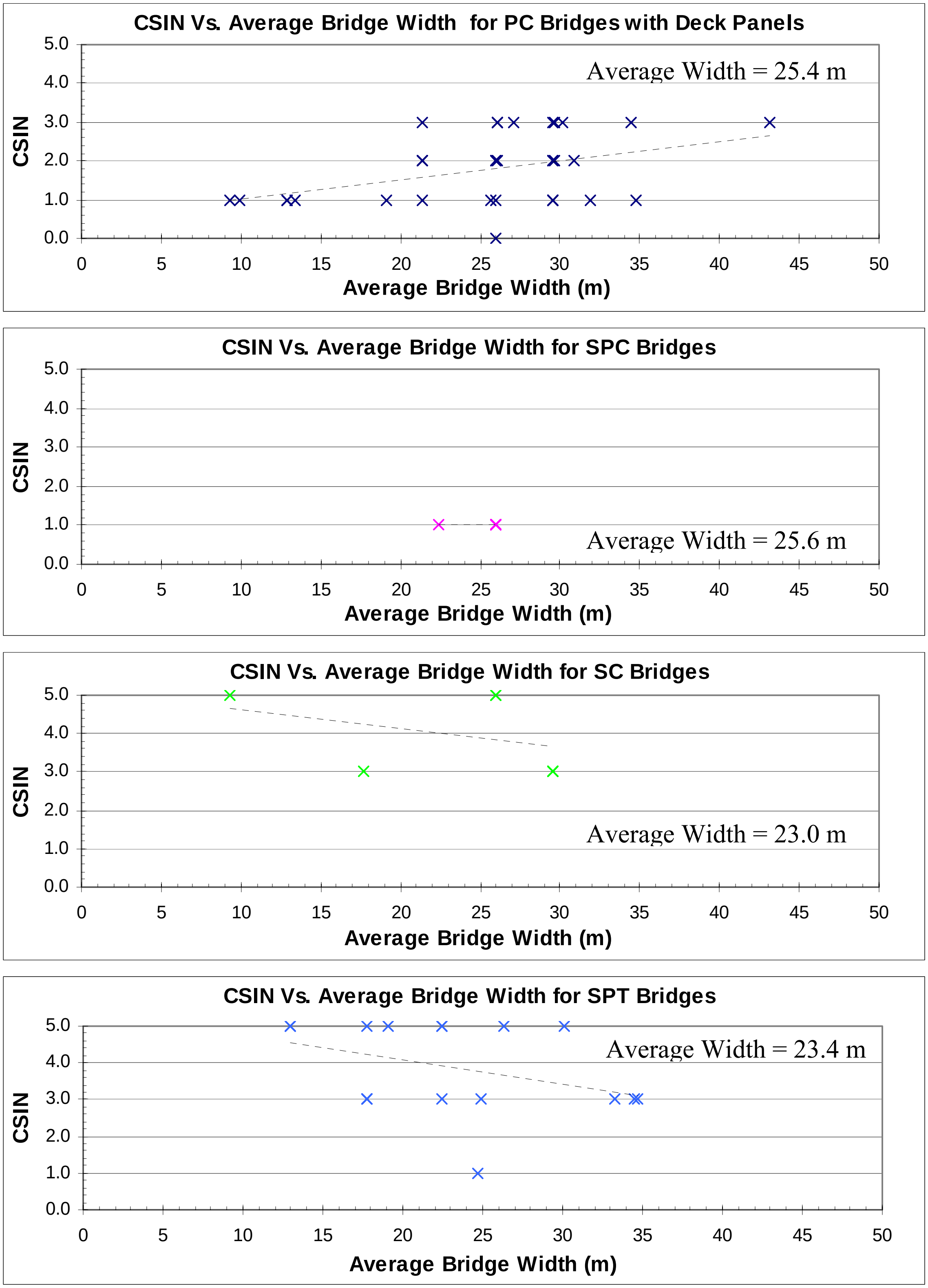


Figure 8 - CSIN vs. average bridge width for the four different bridge types. The trendlines were created using a least squares analysis.

Figure 9 - Photo taken January 8, 2003 of the underside of Bridge No. 50. The longitudinal cracks are radiating from the north integral abutment.

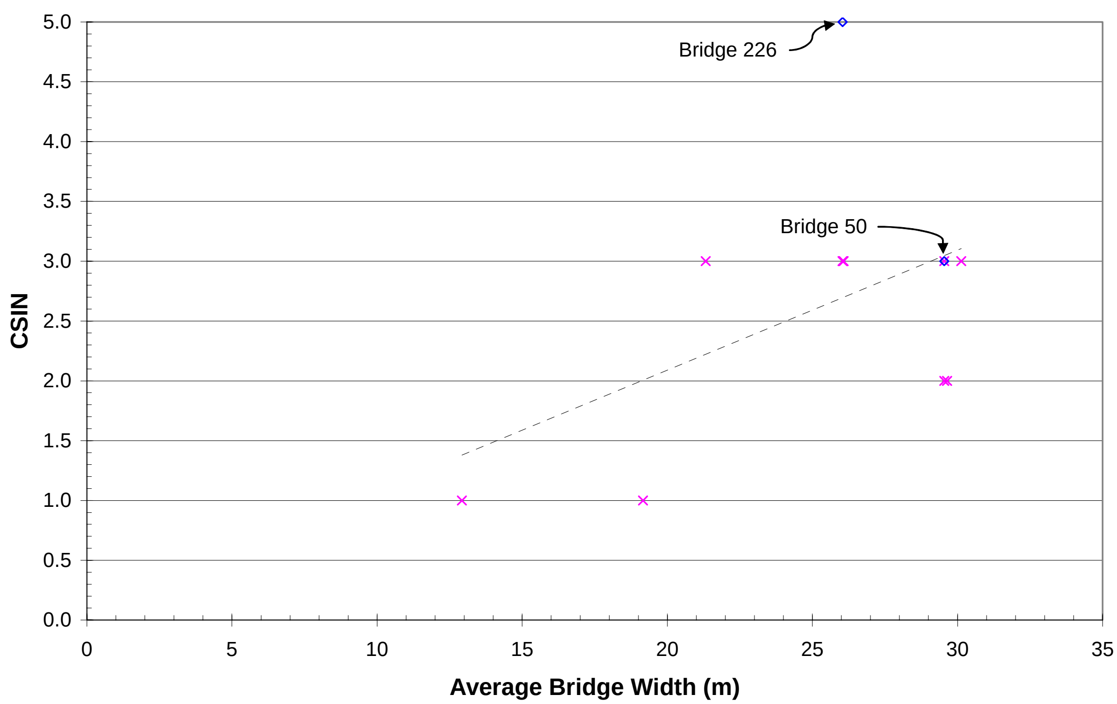


Figure 10 - Chart showing the relationship between bridge width and CSIN ranking for all studied bridges with longitudinal cracks. All bridges are PC bridges with panels except Bridge No. 226 which does not have panels and Bridge No. 50 which is an SC bridge. The trendline was created using a least squares analysis.

8.9 Precast Deck Panels

Precast concrete deck panels were used in the construction of 92% of the concrete girder bridges and none of the steel girder bridges. These panels work as stay-in-place concrete forms. All panels over 1600 mm (5'-3") were prestressed during construction. Once the panels are positioned on the girders, a traditional reinforced concrete cast-in-place slab is placed on top. This composite slab system works well to reduce through cracks in bridge decks (Abendroth, 1995).

Many cracks in concrete bridge decks are caused by restrained shrinkage of the concrete slab during the curing process. When a composite slab system is used, the lower panels have already cured and gone through the shrinkage process when the upper cast-in-place portion of the deck is placed. Therefore, the shrinkage cracks that are observed on the topsides of a composite deck generally do not propagate through the entire deck. Through cracks tend to appear in the bridge deck at the precast panel joints when the bridge is heavily loaded (Abendroth, 1995). This is due to the rapid change in cross-section and the associated stress concentrations that develop at the panel joints.

The corridor standard details for precast concrete deck panels give a range of panel thickness from 90mm (3.5in) to 145mm (5.7in). Bridge decks constructed with precast deck panels range in thickness from 191mm (7.5in) to 318mm (12.5in). Properly reinforced 7.5" thick concrete slabs are generally capable to span between girders without any problems.

As shown in Table 7, only four concrete girder bridges were constructed without precast concrete deck panels. It is not a coincidence that these four bridges are the only concrete girder bridges that received CSIN rankings of 5.

Table 7 - Concrete girder bridges without precast deck panels.

RFP Bridge Number	Type of Structure	Location	CSIN
70	PC	I-15 NB over UPRR at 3600 South	5
72	PC	I-15 SB over UPRR at 3600 South	5
224	PC	I-15 NB over 300 N	5
226	PC	I-15 SB over 300 N	5

Bridge No. 70 and Bridge No. 72 are very similar bridges. Both bridges are skewed 60 degrees. There are no concrete girder bridges with larger skews and only Bridge No. 200, which is 98.25 m (322.3 ft) long, is longer. Bridge No. 70 and Bridge No. 72 are 95.6 m (313.6 ft) and 95.8 m (314.3 ft) long respectively. Precast panels were not used on these bridges because of the complexity of high-skew bridge deck attachment to abutments and bents.

Bridge No. 224 was built in two separate phases. The original portion of the bridge was constructed with precast deck panels. About seven months after the initial deck was constructed, additional lanes were added to the west side of the bridge. This widened portion of the bridge was built without deck panels. From the underside of the bridge, an interesting contrast can be seen between the precast panel portion of the deck and the cast-in-place portion. As seen in Figure 11, the west deck section has regularly spaced transverse cracks that are spaced approximately twice as close as the east side panel joints. During rain storms, water has been observed on the underside of both sections of the deck on multiple occasions. On the east section of the bridge, the water was observed at the precast deck panel joints. On the west section the water was observed at many of the transverse cracks

Figure 11 - Photo of the underside of Bridge No. 224 taken Jan. 4, 2003. The east portion of the deck (left side of photo) was constructed with precast panels and the west portion was cast-in-place.

According to the reviewed construction documents, panels were constructed for Bridge No. 226 but did not meet specification. As a result, a Field Design Change (FDC) Memo was issued and the contractor constructed the deck without panels.

8.10 Comparison with Preliminary UDOT Data

The database was used to compare the preliminary UDOT inspection data to the data obtained from the recent bridge deck inspections.

8.10.1 Cracks After Traffic Loading

The notes contained in the UDOT "Preventive Needs for I-15 Decks" spreadsheet indicate that eleven bridges had not been opened to traffic at the time of the inspection. When compared to the initial UDOT rankings, all eleven of these bridges received a higher or equal cracking severity index ranking when inspected for this study. This indicates that traffic loading increases the severity of bridge deck cracking. This is due to combining wheel load flexural stresses with any residual shrinkage stresses that may exist in the deck.

8.10.2 Surface Cracks Vs. Thru Cracks

A number of bridges received adjusted rankings of 3, 4, or 5 from the UDOT "Preventive Needs" inspections. For fifteen of these bridges, the underside inspection performed for this study yielded CSIN rankings of 1 or 2. Since it is impossible for the cracks to mend themselves with time, it is deduced that much of the topside cracking that was observed by UDOT does not carry through the entire deck. Fourteen of these fifteen bridges have precast concrete deck panels as part of the composite deck system.

9. ECONOMIC CONSIDERATIONS

It is difficult and expensive to place large amounts of minimally cracked concrete. Experience and research has shown that it can be done, but only with the strictest attention to detail (Morin et al., 2002; Waszczuk, 1999). This attention to detail and procedure makes a project more expensive and time consuming. A life cycle cost comparison should be conducted between intricately designed and built structures with minimal deck cracking and structures with more cracks and higher long-term maintenance demands.

9.1 Bridge Deck Overlay Systems

The costs of achieving a crack-free bridge deck may be prohibitive. Instead of undertaking extraordinary measures to eliminate deck cracking, it may be more feasible to accept the fact that bridge decks will crack, and include an overlay in the initial construction to protect them from further deterioration. This is not to suggest that the level of cracking should not be minimized by utilizing simple, less costly procedures and design techniques. But, a correctly applied overlay system will bridge any small deck cracks and keep water from reaching the concrete reinforcing steel. Consideration must be made to account for the necessity of new overlay applications at regular intervals throughout the life of the bridge.

If a deck overlay system was originally planned during the design phase of a bridge, some of the items that contribute to deck cracking become less significant. Some items that may not require as much care and consideration are:

- Girder type;
- Abutment type and restraint;
- Concrete mix design;
- Deck placement sequence or size;
- Curing procedures.

A careful study considering the economic issues pertaining to these variables should be conducted. The Virginia Department of Transportation applied latex modified concrete (LMC) overlays to bridge decks in 1997 with an average cost of \$56.13/m^2 (\$5.22/ft^2). When traffic control was required, the cost increased to \$119.35/m^2 (11.10/ft^2) (Pyć et al., 2000). UDOT generally pays around \$53.82/m^2 (\$5.00/ft^2) for polymer overlays that typically last around 10 years. This is in line with the numbers presented by Yunovich and Thompson (2003) for bridge deck overlay costs.

9.2 Overlay System Vs. Concrete Sealer

The I-15 Reconstruction Project contract required the contractor to apply a silane based sealer to all bridge decks. This type of sealer is designed to inhibit moisture and chlorides from penetrating the concrete and gaining access to the embedded reinforcing steel. This type of sealer works well unless the concrete cracks, which allows water to flow through the crack and bypass the sealer protection barrier. If a polymer membrane overlay is applied to the concrete, the membrane characteristics allow it to bridge small cracks and eliminate water invasion. Large cracks should be epoxy grouted prior to the membrane application. The appropriate use of an overlay system negates the necessity of applying concrete sealers to the bridge decks. These savings can be used to offset some of the costs associated with the installation of a bridge deck overlay system.

10. CONCLUSIONS AND RECOMMENDATIONS

Bridge deck cracking is a major problem across the country (Xi et al., 2003; Yunovich and Thompson, 2003). There are many different factors that contribute to the cracking problem. Design considerations, construction methods, and maintenance practices will all affect bridge deck deterioration. Initially, most cracks do not indicate structural deficiencies. However, most cracks will eventually lead to bridge deck deterioration if they are ignored. The following conclusions and recommendations provide guidance for bridge design engineers to minimize cracking and reduce long term deterioration of bridge decks. These conclusions and recommendations were generated from information obtained from the literature review and the observations made for this study. Following each item below, it is noted where the information was acquired; Literature Review (LR) or Study Observations (SO).

1. Silica fume has been successfully used in concrete bridge decks and has shown ability to reduce chloride infiltration (Lafave et al., 2002). However, the use of silica fume concrete requires stringent construction practices for finishing and curing as outlined in Section 4.3 (Hadidi and Saadeghvaziri, 2003). (LR)

2. Bridge deck cracking is influenced by the size of concrete placements. This can be seen in bridge decks that were constructed using large volumes of concrete placed in a single operation. The trend is particularly noticeable for bridge decks with extremely large width and/or length dimensions. One of the most severely

cracked bridge decks is Bridge No. 230. This five span SC structure is 246 m (807 ft) long and the deck was placed in a single operation. (LR & SO)

3. The size of bridge deck placements should be limited by drying shrinkage considerations. The ideal situation would be to first place concrete in areas of greatest positive moment on shored girders. The shoring would then be removed prior to placing the concrete in negative moment areas. This would limit the tensile stresses in the negative area and pre-compress the deck in the positive regions (Hadidi and Saadeghvaziri, 2003; Xi et al., 2003). Cracking patterns observed in the I-15 Reconstruction Project support this idea. (LR & SO)

4. The use of "trial" placements of silica fume concrete gives the contractor an opportunity to hone his placing and curing skills before placing any actual bridge decks (ACI Committee 234, 1996; Waszczuk, 1999). (LR)

5. There are full depth cracks on nearly all of the new bridges of I-15. These cracks resulted from placing large amounts of deck concrete in constrained environments. (LR & SO)

6. The concrete decks were restrained by composite attachment to girders, bents, diaphragms, and abutments. The rigid attachment between these elements and the deck is essential for economical girder design and seismic load resistance. However, this rigid attachment leads to transverse and diagonal cracking as the concrete cures and shrinks (Xi et al., 2003). (LR & SO)

7. The use of less composite action between girders and bridge decks could help to reduce cracking. Non-uniform placement of shear connectors coupled with segmental placing of deck concrete would reduce transverse cracking. However, less rigid attachment of the bridge deck to the girders would generally require improved girder bending capacity. (SO)

8. In wide decks, longitudinal cracks (cracks parallel to girders) may result from restrained transverse shrinkage. This type of crack was not a significant problem on the study set of bridges. None of the transversely post-tensioned bridge decks in this study had longitudinal cracking. Transversely post-tensioning the deck helps to close any longitudinal cracks but may contribute to diagonal cracking at the abutments and bents. (LR & SO)

9. As the stress in concrete bridge decks due to shrinkage is coupled with the continuity effects of a multi-span system (variations due to moment distribution), the likeliness of transverse cracks occurring in the deck is even greater. These cracks are particularly common in bridge decks near interior support lines due to the additional restraint provided by the supports and the tensile stresses in the deck due to the negative moment (Xi et al., 2003). (LR & SO)

10. The use of pourback strips, as illustrated in Figure 12, could help to reduce the diagonal shrinkage cracks that have been observed near abutments and other transversely stiff elements on many of the studied bridges. Pourback strips allow most of the deck concrete to cure and shrink before the attachment to stiff, restraining elements is made. Pourback strips should be the final deck concrete placed and should be placed in an order similar to Figure 12. The suggested proportions keep the length-to-width ratio of a pourback segment to a maximum value of two. A 72-hour delay should be allowed between the placements of pourback segments. Additional reinforcement placed perpendicular to the typical diagonal cracking pattern within the pourback region will help to keep any cracks closed if they form. (SO)

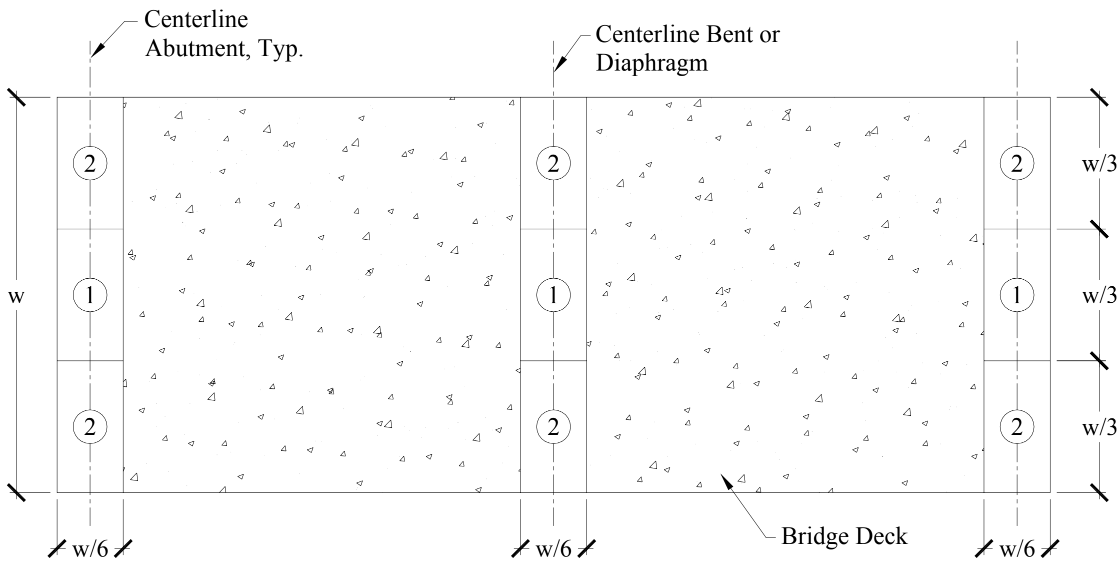


Figure 12 - Recommended placement sequence and size for bridge deck pourback strips.

11. Precast concrete deck panels have worked well on I-15 and other projects to limit the amount of through cracking in bridge decks. In the worst case, precast panels define predictable vertical planes for full-depth cracking to take place. For future concrete girder bridges, the use of a composite bridge deck system consisting of precast concrete panels below a reinforced cast-in-place slab should be considered. (LR & SO)

12. There were no steel girder bridges constructed with precast concrete deck panels in the I-15 Reconstruction Project. A study should be undertaken to determine if this type of construction (steel girders and precast concrete deck panels) is feasible. There is no obvious reason why this combination of elements would not provide a quality bridge deck. Transverse cracking would likely be constrained to the panel joints which is an improvement from much of the cracking observed in this study. These cracks would be in a straight line and would be relatively easy to repair. (SO)

13. Bridge decks on steel girders crack more than decks on concrete girders (Hadidi and Saadeghvaziri, 2003). This is partially due to the generally larger dimensions of decks on steel girders. It is also a function of the use of precast deck panels on 91% of the concrete girder bridges studied. The different thermal expansion coefficients for concrete and steel also contribute to the cracks. Cracking will increase because the steel girders experience a greater increase in length then the concrete deck for the same temperature increase. (LR & SO)

14. The transverse post-tensioning of the SPT bridges may also cause diagonal deck cracking near abutments, bents and rigid diaphragms. This is due to the strain differentials that develop around these stiff elements. This cracking is compounded on skewed bridges where the transverse post-tensioning is carried across a skewed bent or diaphragm. (SO)

15. For decks with transverse post-tensioning, post-tensioning in the same direction could also be applied to the abutments or bents to create elastic shortening of these elements. This shortening would minimize any strain differential and reduce the potential for diagonal deck cracking. (SO)

16. The longitudinal post-tensioning that is present in the SPC bridge girders tends to close up any transverse cracks that may form in the concrete bridge deck (Lounis and Mirza, 1997). These single-span bridges also have an advantage with no negative moment regions in the deck. (LR & SO)

17. Longitudinal post-tensioning of bridge decks should be studied as a means of reducing transverse shrinkage cracks. Any additional downward force imparted to the girder by the post-tensioning strands should be accounted for in the girder design. (SO)

18. Relatively crack free bridge decks can be constructed (Morin et al., 2002). Bridge No. 149 is a large multi-span steel girder structure with a CSIN of 1. A survey of over 50 non I-15 Reconstruction Project bridges indicates that bridge decks have been previously constructed along the Wasatch Front with minimal amounts of cracking. This survey also reveals that other relatively new bridge decks have cracking consistent with that observed on the I-15 project. (LR & SO)

19. The specification and use of membrane overlay systems on bridge decks would simplify deck design and construction procedures because the consequences of minor deck cracking are reduced. Overlay systems help to protect the reinforcing steel from long-term corrosion because they prohibit water and chlorides from entering the concrete. A life-cycle cost analysis should be conducted to determine the efficiency of applying overlays to new bridge decks. (LR & SO)

20. Large cracks should generally be routed and epoxied before overlay application. Additional studies should be conducted to determine the optimal type of overlay system for use on UDOT bridge decks. (SO)

Bridge decks with minimal cracking can be built. This study has pointed out numerous causes of concrete bridge deck cracking and suggested ways to minimize it. This study has also provided to UDOT a working database consisting of the various parameters of the 71 studied bridges along with many digital photos depicting the current state of cracking that is present on each bridge deck.

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11. APPENDIX

11.1 Appendix A - Photos of all 71 Bridges in the Study


North Abutment (Feb. 1, 2003)


North Abutment (March 18, 2003)


Deck Underside (Jan. 2, 2003)


South Abutment (March 18, 2003)


South Abutment (Jan. 2, 2003)


North Abutment (March 18, 2003)

Figure A1 – Typical Photos of Bridge #3, I-15 South Bound over 10000 South.


West Elevation (Dec. 20, 2002)


North Abutment (Jan. 2, 2003)


Abutment Diaphragm (Dec. 20, 2002)


South Abutment (Dec. 20, 2002)


North Abutment (Jan. 2, 2003)


North Abutment (March 18, 2003)

Figure A2 – Typical Photos of Bridge #3.5, I-15 North Bound over 10000 South.


West Elevation (Dec. 20, 2002)


South Abutment (Jan. 8, 2003)


South Abutment (Dec. 20, 2002)


South Abutment (Jan. 8, 2003)


Deck Underside (Jan. 8, 2003)


North Abutment (March 18, 2003)

Figure A3 – Typical Photos of Bridge #8, I-15 North Bound over 9000 South.

East Elevation (Dec. 20, 2002)

West Side of North Abutment (March 18, 2003)

Deck Panel Joint on Underside (Jan. 8, 2003)

South Abutment (Jan. 8, 2003)

Deck Underside (Jan. 8, 2003)

South Abutment (May 21, 2003)

Figure A4 – Typical Photos of Bridge #10, I-15 South Bound over 9000 South.

West Elevation (Dec. 20, 2002)

East Side of North Bent (Feb. 1, 2003)

South Abutment (Dec. 20, 2002)

South Abutment (May 21, 2003)

East Side of North Abutment (Feb. 1, 2003)

North Abutment (May 21, 2003)

Figure A5 – Typical Photos of Bridge #12, I-15 North Bound over Wasatch Street.

Underside Looking South (Jan. 2, 2003)

North Abutment (May 21, 2003)

Deck Underside (Jan. 2, 2003)

South Abutment (May 21, 2003)

South Abutment (Feb. 1, 2003)

South Abutment (May 21, 2003)

Figure A6 – Typical Photos of Bridge #14, I-15 South Bound over Wasatch Street.


West Elevation (Dec. 20, 2002)


Deck Underside (Dec. 20, 2002)


South Abutment (Dec. 20, 2002)


Bridge Underside (Jan. 2, 2003)


Deck Underside (Dec. 20, 2002)


North Side of North Bent (May 21, 2003)

Figure A7 – Typical Photos of Bridge #16, I-15 North Bound over Center Street.

Concrete Girder (Jan. 2, 2003)

South Side of South Bent (May 21, 2003)

Underside Looking North (Feb. 1, 2003)

South Side of South Bent (May 21, 2003)

Figure A8 – Typical Photos of Bridge #18, I-15 South Bound over Center Street.


West Elevation (Dec. 20, 2002)


South Abutment (Jan. 28, 2003)


South Abutment (Dec. 20, 2002)


Underside Looking South (Feb. 1, 2003)


Deck Underside (Jan. 8, 2003)


South Abutment (May 21, 2003)

Figure A9 – Typical Photos of Bridge #20, I-15 North Bound over 7200 South.

East Elevation (Dec. 20, 2002)

South Abutment (Jan. 8, 2003)

Deck Underside (Jan. 8, 2003)

North Abutment (Feb. 1, 2003)

Deck Underside (Jan. 8, 2003)

South Abutment (May 21, 2003)

Figure A10 – Typical Photos of Bridge #22, I-15 South Bound over 7200 South.

Partial West Elevation (Dec. 20, 2002)

South Abutment (Jan. 8, 2003)

South Abutment (Dec. 20, 2002)

Deck Underside (Jan. 8, 2003)

Deck Underside (Jan. 8, 2003)

South Abutment (Jan. 28, 2003)

Figure A11 – Typical Photos of Bridge #23, I-15 North Bound to I-215 West Bound Ramp over 7200 South.


North Abutment (March 18, 2003)


Deck Underside (March 18, 2003)


Deck Underside (March 18, 2003)


Deck Underside (March 18, 2003)


Deck Underside (March 18, 2003)


South Abutment (March 18, 2003)

Figure A12 – Typical Photos of Bridge #26, I-15 North Bound over UPRR at Approximately 6700 South.

East Elevation (Dec. 30, 2002)

Deck Underside (March 18, 2003)

Girder Connection to North Abutment (March 18, 2003)

North Abutment (July 8, 2003)

Underside near North Abutment (March 18, 2003)

Deck Underside (July 8, 2003)

Figure A13 – Typical Photos of Bridge #27, I-215 East Bound to I-15 South Bound Ramp over UPRR at Approximately 6700 South.


Deck Underside (March 18, 2003)


Deck Underside (March 18, 2003)


Deck Underside (March 18, 2003)


Deck Underside (July 8, 2003)


South Abutment (March 18, 2003)


North Side of North Bent (July 8, 2003)

Figure A14 – Typical Photos of Bridge #28, I-15 South Bound over UPRR at Approximately 6700 South.


East Elevation (Dec. 30, 2002)


South Abutment (May 21, 2003)


South Abutment (Dec. 30, 2002)


North Side of North Bent (May 21, 2003)

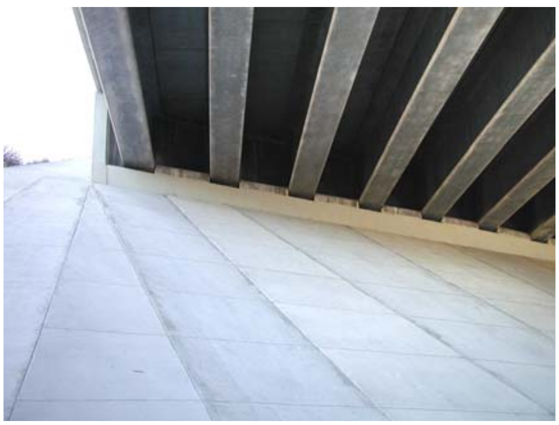

East Side of South Abutment (March 18, 2003)


North Abutment (May 21, 2003)

Figure A15 – Typical Photos of Bridge #29, I-215 East Bound to I-15 South Bound Ramp over UTA RR at Approximately 6700 South.

West Elevation (Dec. 30, 2002)

North Abutment (March 18, 2003)



South Abutment (Dec. 30, 2002)

North Abutment (March 18, 2003)

North Abutment (March 18, 2003)

South Abutment (March 18, 2003)

Figure A16 – Typical Photos of Bridge #30, I-15 North Bound over UTA RR at Approximately 6700 South.

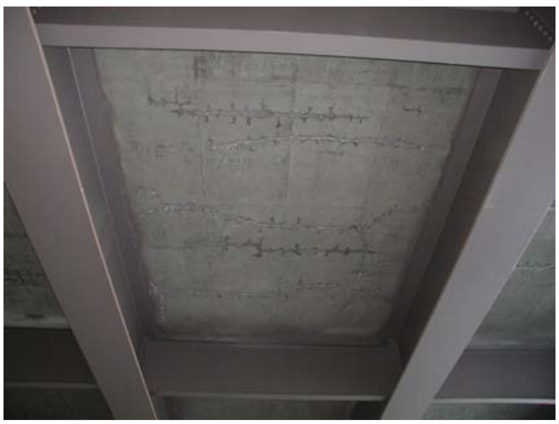

South Abutment (March 18, 2003)


South Abutment (March 18, 2003)

Figure A17 – Typical Photos of Bridge #32, I-15 South Bound over UTA RR at Approximately 6700 South.


West Elevation (Dec. 17, 2002)


Deck Underside (Jan. 10, 2003)


Deck Underside (Jan. 8, 2003)


South Abutment (Jan. 10, 2003)


North Abutment (Jan. 8, 2003)


Deck Underside (Jan. 28, 2003)

Figure A18 – Typical Photos of Bridge #50, I-15 North Bound over 5900 South.

East Elevation (Dec. 17, 2002)

Deck Underside (Jan. 10, 2003)

South Abutment (Jan. 8, 2003)

Deck Underside (Jan. 28, 2003)

North Abutment (Jan. 10, 2003)

South Abutment (Jan. 28, 2003)

Figure A19 – Typical Photos of Bridge #52, I-15 South Bound over 5900 South.

West Elevation (Dec. 17, 2002)

Underside Looking South (Jan. 8, 2003)

North Abutment (Dec. 17, 2002)

North Abutment (Jan. 28, 2003)

Deck Underside (Jan. 8, 2003)

South Abutment (May 21, 2003)

Figure A20 – Typical Photos of Bridge #54, I-15 North Bound over 5300 South.


East Elevation (Dec. 17, 2002)


North Abutment (Jan. 28, 2003)


Deck Underside (Jan. 8, 2003)


South Abutment (May 21, 2003)


North Abutment (Jan. 8, 2003)

Figure A21 – Typical Photos of Bridge #56, I-15 South Bound over 5300 South.


West Elevation (Dec. 17, 2002)


North Abutment (Jan. 28, 2003)


Deck Underside (Jan. 8, 2002)


East Side of South Abutment (Feb. 1, 2003)


North Abutment (Jan. 28, 2003)


North Abutment (Feb. 2, 2003)

Figure A22 – Typical Photos of Bridge #60, I-15 North Bound over 4800 South.


East Elevation (Dec. 17, 2002)


South Abutment (Jan. 28, 2003)


South Abutment (Jan. 8, 2003)


North Abutment (Jan. 28, 2003)


South Abutment (Jan. 8, 2003)


South Abutment (Jan. 28, 2003)

Figure A23 – Typical Photos of Bridge #62, I-15 South Bound over 4800 South.


West Elevation (Dec. 17, 2002)


North Abutment (Jan. 28, 2003)


Deck Underside Looking South (Jan. 13, 2003)


North Abutment (Feb. 25, 2003)

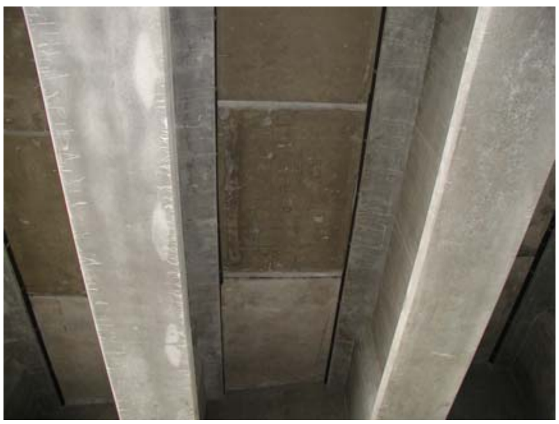

North Abutment (Jan. 13, 2003)

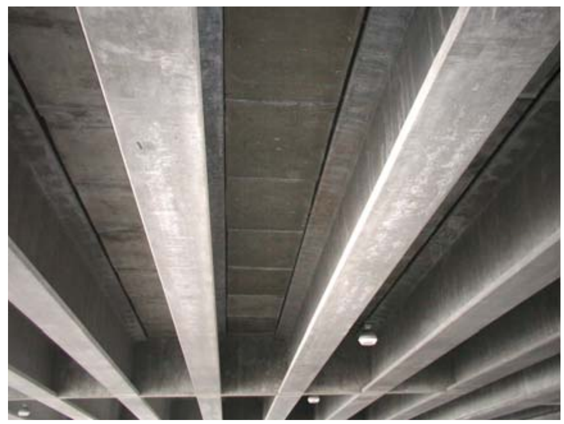

North Abutment (Feb. 25, 2003)

Figure A24 – Typical Photos of Bridge #64, I-15 North Bound over 4500 South.


East Elevation (Dec. 17, 2002)


South Abutment (Feb. 1, 2003)


Deck Underside (Jan. 13, 2003)


Deck Underside Looking South (Feb. 1, 2003)


North Abutment (Jan. 13, 2003)


North Abutment (Feb. 25, 2003)

Figure A25 – Typical Photos of Bridge #66, I-15 South Bound over 4500 South.


West Elevation (Dec. 30, 2002)


Deck Underside (July 8, 2003)


Deck Underside (Jan. 28, 2003)


South Abutment (July 8, 2003)


North Abutment (July 8, 2003)


Deck Underside (July 8, 2003)

Figure A26 – Typical Photos of Bridge #70, I-15 North Bound over UPRR at Approximately 3500 South.


Deck Underside (Dec. 30, 2002)


North Side of South Bent (Jan. 28, 2003)


South Abutment (Jan. 28, 2003)


Deck Underside (July 8, 2003)


Deck Underside (Jan. 28, 2003)


South Side of North Bent (July 8, 2003)

Figure A27 – Typical Photos of Bridge #72, I-15 North Bound over UPRR at Approximately 3500 South.

West Elevation (Dec. 17, 2002)

South Abutment (Dec. 17, 2002)

South Abutment (May 21, 2003)

Figure A28 – Typical Photos of Bridge #74, I-15 North Bound over 3300 South.

East Elevation (Dec. 17, 2002)

South Abutment (May 21, 2003)

South Abutment (Dec. 17, 2002)

South Abutment (Jan. 28, 2003)

Figure A29 – Typical Photos of Bridge #76, I-15 South Bound over 3300 South.


West Abutment (March 18, 2003)


East Abutment (March 18, 2003)


West Abutment (March 18, 2003)


East Abutment (March 18, 2003)

Figure A30 – Typical Photos of Bridge #112, I-80 West Bound to SR 201 West Bound over UTA RR.


East Abutment (March 18, 2003)


West Abutment (March 18, 2003)


East Abutment (March 18, 2003)


West Abutment (Jan. 8, 2003)


Deck Underside Looking West (March 18, 2003)

Figure A31 – Typical Photos of Bridge #114, I-80 West Bound over UTA RR.


North Elevation (Dec. 30, 2002)


Deck Underside Looking West (March 18, 2003)


East Abutment (March 18, 2003)


West Abutment (March 18, 2003)


East Abutment (March 18, 2003)


West Abutment (March 18, 2003)

Figure A32 – Typical Photos of Bridge #116, I-80 East Bound over UTA RR.


West Elevation (Dec. 17, 2002)


South Abutment (May 21, 2003)


North Abutment (Dec. 17, 2002)


South Abutment (May 21, 2003)


South Abutment (Feb. 14, 2003)


South Abutment (May 21, 2003)

Figure A33 – Typical Photos of Bridge #138, I-15 North Bound Collector over 2100 South.

West Elevation (Dec. 17, 2002)

South Abutment (April 15, 2003)

East Side of South Abutment (Feb. 14, 2003)

South Abutment (April 15, 2003)

South Abutment (April 15, 2003)

South Abutment (May 21, 2003)

Figure A34 – Typical Photos of Bridge #140, I-15 North Bound over 2100 South.


West Elevation (Dec. 17, 2002)


Deck Underside at South Abutment (May 21, 2003)


Underside Looking South (Dec. 17, 2002)


South Abutment (Feb. 14, 2003)

Figure A35 – Typical Photos of Bridge #142, I-15 South Bound Collector over 2100 South.


West Elevation (Dec. 17, 2002)


South Abutment (April 15, 2003)


South Abutment (Feb. 14, 2003)


South Abutment May 21, 2003)


South Underside (Feb. 14, 2003)


South Abutment (May 21, 2003)

Figure A36 – Typical Photos of Bridge #144, I-15 South Bound over 2100 South.

West Elevation (Dec. 17, 2002)

North Abutment (Feb. 13, 2003)

East Side of South Abutment (Feb. 13, 2003)

Deck Underside (Feb. 13, 2003)

North Abutment (Feb. 13, 2003)

South Abutment (Feb. 13, 2003)

Figure A37 – Typical Photos of Bridge #145, I-15 North Bound over Andy Avenue.


West Elevation (Dec. 17, 2002)


North Abutment (Feb. 13, 2003)


South Abutment (Dec. 17, 2002)


South Abutment (Feb. 13, 2003)


North Abutment (Feb. 13, 2003)


South Abutment (Feb. 13, 2003)

Figure A38 – Typical Photos of Bridge #147, I-15 South Bound over Andy Avenue.


West Elevation (Dec. 17, 2002)


South Abutment (Feb. 14, 2003)


Deck Underside (Jan. 13, 2003)


South Abutment (April 15, 2003)


North Abutment (Feb. 14, 2003)


South Abutment (April 15, 2003)

Figure A39 – Typical Photos of Bridge #148, I-15 North Bound Collector over 1700 South.

Partial West Elevation (Feb. 13, 2003)

Deck Underside (July 8, 2003)

North Abutment (Feb. 13, 2003)

Deck Underside Looking South (July 8, 2003)

Deck Underside just North of the 2nd Bent from the North (Feb. 13, 2003)

North Abutment (July 8, 2003)

Figure A40 – Typical Photos of Bridge #149, I-15 North Bound Collector over Andy Avenue.


East Elevation (Dec. 17, 2002)


South Abutment (Feb. 14, 2003)


Deck Underside (Jan. 13, 2003)


South Abutment (April 15, 2003)


North Abutment (Feb. 14, 2003)


South Abutment (April 15, 2003)

Figure A41 – Typical Photos of Bridge #150, I-15 South Bound Collector over 1700 South.

West Elevation (Dec. 17, 2002)

Deck Underside (Feb. 13, 2003)

South Abutment (Dec. 17, 2002)

North Abutment (Feb. 13, 2003)

North Abutment (Jan. 13, 2003)

South Abutment (Feb. 13, 2003)

Figure A42 – Typical Photos of Bridge #151, I-15 South Bound to I-80 East Bound over Andy Avenue.


Underside Looking South (Dec. 17, 2002)


North Abutment (Feb. 14, 2003)


Deck Underside (Jan. 13, 2003)


South Abutment (Feb. 14, 2003)


North Abutment (Feb. 14, 2003)


South Abutment (April 15, 2003)

Figure A43 – Typical Photos of Bridge #152, I-15 North Bound over 1700 South.


West Elevation (Dec. 17, 2002)


North Abutment (Feb. 13, 2003)


Deck Underside Looking North (Jan. 13, 2003)


North Abutment (Feb. 13, 2003)


North Abutment (Feb. 13, 2003)


Underside near North Abutment (Feb. 13, 2003)

Figure A44 – Typical Photos of Bridge #153, I-15 South Bound Collector over Andy Avenue.


North Abutment (Feb. 14, 2003)


South Abutment (Feb. 14, 2003)


North Abutment (Feb. 14, 2003)


Underside near South Abutment (April 15, 2003)


South Abutment (Feb. 14, 2003)


South Abutment (April 15, 2003)

Figure A45 – Typical Photos of Bridge #154, I-15 South Bound over 1700 South.

East Elevation (Dec. 17, 2002)

Deck Underside (Jan. 13, 2003)

South Abutment (Dec. 17, 2002)

East Side of Bent (Feb. 13, 2003)

South Side of Bent (Dec. 17, 2002)

South Side of Bent (Feb. 13, 2003)

Figure A46 – Typical Photos of Bridge #155, Collector Ramp to I-15 South Bound over Andy Avenue.

Partial West Elevation (Dec. 17, 2002)

East Side of South Bent (Feb. 25, 2003)

South Abutment (Dec. 17, 2002)

South Abutment (April 15, 2003)

Deck Underside Looking North (Dec. 17, 2002)

South Abutment (April 15, 2003)

Figure A47 – Typical Photos of Bridge #156, I-15 North Bound Exit Ramp to 900 South over 1300 South.

East Elevation (Dec. 17, 2002)

South Abutment (Feb. 25, 2003)

South Abutment (Feb. 25, 2003)

Deck Underside near South Abutment (April 15, 2003)

South Abutment (Feb. 25, 2003)

South Abutment (April 15, 2003)

Figure A48 – Typical Photos of Bridge #158, I-15 South Bound Collector over 1300 South.

West Elevation (Dec. 17, 2002)

South Abutment (Feb. 25, 2003)

South Abutment (Dec. 17, 2002)

Deck Underside just South of South Bent (Feb. 25, 2003)

South Abutment (Feb. 25, 2003)

Deck Underside just South of South Bent (Feb. 25, 2003)

Figure A49 – Typical Photos of Bridge #160, I-15 North Bound over 1300 South.


West Side of Bridge (Dec. 17, 2002)


South Abutment (Feb. 25, 2003)


South Abutment (Feb. 25, 2003)


Deck Underside (Feb. 25, 2003)


South Abutment (Feb. 25, 2003)


South Abutment (April 15, 2003)

Figure A50 – Typical Photos of Bridge #162, I-15 South Bound over 1300 South.


West Elevation (Dec. 21, 2002)


South Abutment (Feb. 25, 2003)


Bridge Underside (Jan. 10, 2003)


Deck Underside just South of South Bent (Feb. 25, 2003)


North Abutment (Jan. 10, 2003)


Deck Underside Looking North (Feb. 25, 2003)

Figure A51 – Typical Photos of Bridge #168, I-15 North Bound over 500 West and UPRR.


Deck Underside (Jan. 10, 2003)


South Abutment (Feb. 25, 2003)


Deck Underside (Jan. 10, 2003)


South Abutment (Feb. 25, 2003)


North Abutment (Feb. 25, 2003)


Deck Underside just South of South Bent (Feb. 25, 2003)

Figure A52 – Typical Photos of Bridge #170, I-15 South Bound over 500 West and UPRR.

Partial West Elevation (Dec. 21, 2002)

Deck Underside (Jan. 4, 2003)

Deck Underside (Jan. 4, 2003)

Deck Underside just South of North Bent (Feb. 25, 2003)

Deck Underside (Jan. 4, 2003)

Deck Underside (Feb. 25, 2003)

Figure A53 – Typical Photos of Bridge #174, I-15 North Bound over 900 South.

East Elevation (Dec. 21, 2002)

Deck Underside (Feb. 25, 2003)

Deck Underside (Jan. 4, 2003)

North Abutment (Feb. 25, 2003)

Deck Underside (Feb. 25, 2003)

Deck Underside Looking South (Feb. 25, 2003)

Figure A54 – Typical Photos of Bridge #176, I-15 South Bound over 900 South.


West Elevation (Dec. 21, 2002)


Deck Underside (Jan. 10, 2003)


Deck Underside (Jan. 4, 2003)


Deck Underside (Jan. 10, 2003)


Deck Underside (Jan. 10, 2003)


North Side of North Bent (May 21, 2003)

Figure A55 – Typical Photos of Bridge #180, I-15 North Bound over 800 South.

Deck Underside near Abutment (Jan. 10, 2003)

Deck Underside near South Bent (May 21, 2003)

Deck Underside near North Abutment (May 21, 2003)

Figure A56 – Typical Photos of Bridge #182, I-15 South Bound over 800 South.


Partial West Elevation (Dec. 21, 2002)


Deck Underside (Jan. 10, 2003)


North Abutment (Dec. 21, 2002)


Deck Underside (Jan. 10, 2003)


Deck Underside (Jan. 10, 2003)


Deck Underside (Feb. 25, 2003)

Figure A57 – Typical Photos of Bridge #196, I-15 North Bound over 400 South.

North Abutment (Dec. 21, 2002)

Deck Underside (Feb. 25, 2003)

Deck Underside (Jan. 10, 2003)

South Abutment (Feb. 25, 2003)

Deck Underside (Jan. 10, 2003)

Deck Underside (Feb. 25, 2003)

Figure A58 – Typical Photos of Bridge #198, I-15 South Bound over 400 South.

West Elevation (Dec. 21, 2002)

North Abutment (Feb. 14, 2003)

North Abutment (Jan. 10, 2003)

South Abutment (Feb. 25, 2003)

East Side of North Bent (Feb. 14, 2003)

Deck Underside (Feb. 25, 2003)

Figure A59 – Typical Photos of Bridge #200, I-15 North Bound to I-80 West Bound Ramp over 400 South.

Partial West Elevation (Dec. 21, 2002)

North Abutment (Jan. 10, 2003)

Deck Underside (Jan. 10, 2003)

East Side of South Bent (Feb. 25, 2003)

Deck Underside (Jan. 10, 2003)

Deck Underside (Feb. 25, 2003)

Figure A60 – Typical Photos of Bridge #202, I-80 East Bound to I-15 South Bound Ramp over 400 South.

West Elevation (Dec. 21, 2002) [the lower bridge]

Deck Underside just South of North Bent (Feb. 14, 2003)

North Abutment (Feb. 14, 2003)

South Abutment (Feb. 14, 2003)

North Abutment (Feb. 14, 2003)

Deck Underside just North of South Bent (Feb. 14, 2003)

Figure A61 – Typical Photos of Bridge #212, I-15 North Bound over 200 South.

East Elevation (Dec. 21, 2002)

Deck Underside Looking South (Feb. 14, 2003)

North Abutment (Feb. 14, 2003)

Deck Underside just North of North Bent (Feb. 14, 2003)

North Abutment (Feb. 14, 2003)

South Abutment (Feb. 14, 2003)

Figure A62 – Typical Photos of Bridge #214, I-15 South Bound over 200 South.


Partial West Elevation (Dec. 21, 2002)


Deck Underside (Jan. 4, 2003)


Deck Underside (Dec. 21, 2002)


North Abutment (Jan. 4, 2003)


Deck Underside (Jan. 4, 2003)


North Abutment (Jan. 4, 2003)

Figure A63 – Typical Photos of Bridge #216, I-15 North Bound over South Temple.


Partial West Elevation (Dec. 21, 2002)


Deck Underside (Jan. 4, 2003)


North Abutment (Dec. 21, 2002)


Deck Underside (Jan. 4, 2003)


Deck Underside (Jan. 4, 2003)


Deck Underside (Jan. 10, 2003)

Figure A64 – Typical Photos of Bridge #218, I-15 South Bound over South Temple.


Partial East Elevation (Dec. 21, 2002)


Deck Underside Looking North (Feb.14, 2003)


North Abutment (Jan. 31, 2003)


North Abutment (Feb. 14, 2003)


Deck Underside just South of North Bent (Feb. 14, 2003)


North Abutment (Feb. 14, 2003)

Figure A65 – Typical Photos of Bridge #220, I-15 North Bound over North Temple.

Partial East Elevation (Dec. 21, 2002)

North Abutment (Feb. 14, 2003)

Deck Underside near North Bent (Jan. 31, 2003)

North Side of North Bent (Feb. 14, 2003)

North Abutment (Jan. 31, 2003)

North Abutment (Feb. 14, 2003)

Figure A66 – Typical Photos of Bridge #222, I-15 South Bound over North Temple.

Deck Underside (Jan. 4, 2003)

Deck Underside (Jan. 10, 2003)

Deck Underside (Jan. 10, 2003)

North Abutment (Jan. 10, 2003)

Deck Underside near North Abutment (Jan. 10, 2003)

South Abutment (Jan. 10, 2003)

Figure A67 – Typical Photos of Bridge #224, I-15 North Bound over 300 North.

East Elevation (Dec. 21, 2002)

Deck Underside Looking South (Jan. 10, 2003)

Deck Underside Looking North (Jan. 4, 2003)

North Abutment (Jan. 10, 2003)

Deck Underside (Jan. 10, 2003)

South Abutment (Jan. 10, 2003)

Figure A68 – Typical Photos of Bridge #226, I-15 South Bound over 300 North.


North-West Abutment (Jan. 31, 2003)


Deck Underside (Jan. 31, 2003)


Deck Underside (Jan. 4, 2003)


Deck Underside (Feb. 25, 2003)


Deck Underside (Jan. 4, 2003)


Deck Underside (Feb. 25, 2003)

Figure A69 – Typical Photos of Bridge #230, I-15 South Bound to I-80 West Bound over 200 South.


Partial West Elevation (Dec. 20, 2002)


Deck Underside (Jan. 28, 2003)


Deck Underside (Jan. 8, 2003)


North Abutment (Feb. 2, 2003)

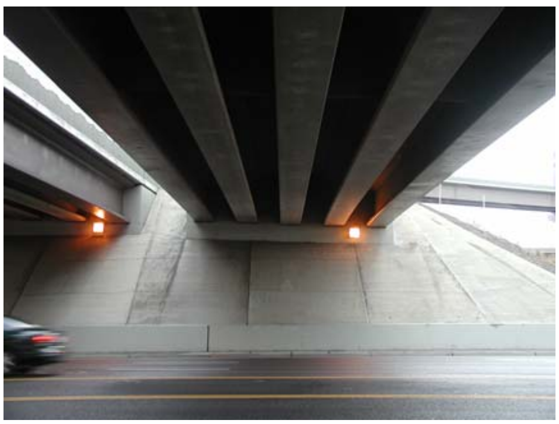

South Abutment (Jan. 28, 2003)


Deck Underside (Feb. 2, 2003)

Figure A70 – Typical Photos of Bridge #702, I-15 South Bound Collector over 7200 South.

East Elevation (Dec. 17, 2002)

South Abutment (Feb. 14, 2003)



Bridge Underside Looking South (Dec. 17, 2002)

South Abutment (Feb. 14, 2003)

Figure A71 – Typical Photos of Bridge #12002, I-15 South Bound Collector over 2100 South.

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11.2 Appendix B - Sample Database Queries

Query 1 - Bridge Decks Constructed Between April 1998 and December 1998

RFP Bridge Number	Type of Structure	Location	Date of First Placement	CSIN
14	PC	I-15 SB over Wasatch St (8000 S)	9-Apr-98	1
18	PC	I-15 SB over 7800 S (Center St)	28-Apr-98	1
702	SPT	I-15 SB CD over 7200 S	10-Jun-98	5
62	PC	I-15 SB over 4800 S	5-Aug-98	2
22	SPC	I-15 SB over 7200 S	7-Aug-98	1
23	SPT	I-15 NB / I-215 WB Ramp over 7200 S	8-Aug-98	5
29	PC	I-215 EB to 7200 S / I-15 SB CD Ramp over UTA RR	17-Aug-98	1
27	SC	I-215 EB to I-15 SB Ramp over UPRR	17-Aug-98	3
66	SPC	I-15 SB over 4500 S	22-Aug-98	1
142	PC	I-15 SB CD over 2100 S	1-Sep-98	1
3	PC	I-15 SB over 10000 S	4-Sep-98	1
144	PC	I-15 SB over 2100 S	14-Sep-98	2
10	SPC	I-15 SB over 9000 S	22-Sep-98	1
12002	PC	I-15 SB CD over 2100 S	2-Oct-98	1
147	SPT	I-15 SB over Andy Ave	23-Oct-98	3
151	SPT	I-15 SB to I-80 EB over Andy Ave	1-Dec-98	3

Query 2 - Bridge Decks Constructed Between January 1999 and September 1999

RFP Bridge Number	Type of Structure	Location	Date of First Placement	CSIN
168	PC	I-15 NB over 500 W and UPRR	24-Feb-99	3
52	SC	I-15 SB over 5900 S	13-Mar-99	3
74	SPC	I-15 NB over 3300 S	15-Mar-99	1
160	PC	I-15 NB over 1300 S	16-Mar-99	2
56	SPC	I-15 SB over 5300 S	17-Mar-99	1
216	SPT	I-15 NB over S Temple	23-Mar-99	5
150	PC	I-15 SB CD over 1700S	7-Apr-99	2
153	PC	I-15 SB CD over Andy Ave	15-Apr-99	1
154	PC	I-15 SB over 1700 S	15-Apr-99	2
32	SPC	I-15 SB over UTA RR	17-Apr-99	1
114	SPT	I-80 WB over UTA RR	17-Apr-99	3
112	SPT	I-80 WB (Ramp) To SR 201 WB over UTA RR	20-Apr-99	5
155	PC	I-15 SB CD to I-15 SB over Andy Ave	26-Apr-99	1
180	PC	I-15 NB over 800 S	26-Apr-99	3
28	SC	I-15 SB over UPRR	1-May-99	5
70	PC	I-15 NB over UPRR	5-May-99	5
174	SPT	I-15 NB over 900 S	17-May-99	3
212	PC	I-15 NB over 200 S	28-May-99	3
220	PC	I-15 NB over N Temple	8-Jun-99	3
224	PC	I-15 NB over 300 N	11-Jun-99	5
196	SPT	I-15 NB over 400 S	25-Jun-99	5
12	PC	I-15 NB over Wasatch St (8000 S)	30-Jun-99	1
3.5	PC	I-15 NB over 10000 S	6-Jul-99	0
148	PC	I-15 NB CD over 1700 S	15-Jul-99	1
16	PC	I-15 NB over 7800 S (Center St)	22-Jul-99	2
138	PC	I-15 NB CD over 2100 S	28-Jul-99	1
149	SPT	I-15 NB CD over Andy Ave	25-Aug-99	1
8	SPC	I-15 NB over 9000 S	1-Sep-99	1

Query 3 - Bridges Constructed Between October 1999 and June 2000

RFP Bridge Number	Type of Structure	Location	Date of First Placement	CSIN
20	SPC	I-15 NB over 7200 S	2-Oct-99	1
54	SPC	I-15 NB over 5300 S	3-Feb-00	1
30	SPC	I-15 NB over UTA RR	15-Feb-00	1
60	PC	I-15 NB over 4800 S	4-Mar-00	3
50	SC	I-15 NB over 5900 S	10-Mar-00	3
76	SPC	I-15 SB over 3300 S	11-Mar-00	1
26	SC	I-15 NB over UPRR	4-Apr-00	5
64	SPC	I-15 NB over 4500 S	17-Apr-00	1
200	PC	Ramp NW over 400 S	20-Apr-00	3
158	PC	I-15 SB CD over 1300 S	2-May-00	2
145	SPT	I-15 NB over Andy Ave	3-May-00	3
162	PC	I-15 SB over 1300 S	5-May-00	2
152	PC	I-15 NB over 1700 S	15-May-00	1
170	PC	I-15 SB over 500 W and UPRR	29-Jun-00	3

Query 4 - Bridges Constructed Between July 2000 and March 2001

RFP Bridge Number	Type of Structure	Location	Date of First Placement	CSIN
116	SPT	I-80 EB over UTA RR	12-Jul-00	3
140	PC	I-15 NB over 2100 S	13-Jul-00	2
202	SPT	Ramp ES over 400 S	27-Jul-00	5
222	PC	I-15 SB over N Temple	3-Oct-00	3
182	PC	I-15 SB over 800 S	13-Oct-00	1
214	PC	I-15 SB over 200 S	3-Nov-00	3
156	PC	I-15 900S-A over 1300 S	21-Nov-00	1
230	SC	Ramp SW over 200 S	16-Dec-00	5
176	SPT	I-15 SB over 900 S	10-Jan-01	3
72	PC	I-15 SB over UPRR	2-Feb-01	5
226	PC	I-15 SB over 300 N	2-Feb-01	5
218	SPT	I-15 SB over S Temple	15-Feb-01	5
198	SPT	I-15 SB over 400 S	7-Mar-01	5

Query 5 – All PC Bridge Structures

RFP Bridge Number	Type of Structure	Location	CSIN
3	PC	I-15 SB over 10000 S	1
3.5	PC	I-15 NB over 10000 S	0
12	PC	I-15 NB over Wasatch St (8000 S)	1
14	PC	I-15 SB over Wasatch St (8000 S)	1
16	PC	I-15 NB over 7800 S (Center St)	2
18	PC	I-15 SB over 7800 S (Center St)	1
29	PC	I-215 EB to 7200 S / I-15 SB CD Ramp over UTA RR	1
60	PC	I-15 NB over 4800 S	3
62	PC	I-15 SB over 4800 S	2
70	PC	I-15 NB over UPRR	5
72	PC	I-15 SB over UPRR	5
138	PC	I-15 NB CD over 2100 S	1
140	PC	I-15 NB over 2100 S	2
142	PC	I-15 SB CD over 2100 S	1
144	PC	I-15 SB over 2100 S	2
148	PC	I-15 NB CD over 1700 S	1
150	PC	I-15 SB CD over 1700S	2
152	PC	I-15 NB over 1700 S	1
153	PC	I-15 SB CD over Andy Ave	1
154	PC	I-15 SB over 1700 S	2
155	PC	I-15 SB CD to I-15 SB over Andy Ave	1
156	PC	I-15 900S-A over 1300 S	1
158	PC	I-15 SB CD over 1300 S	2
160	PC	I-15 NB over 1300 S	2
162	PC	I-15 SB over 1300 S	2
168	PC	I-15 NB over 500 W and UPRR	3
170	PC	I-15 SB over 500 W and UPRR	3
180	PC	I-15 NB over 800 S	3
182	PC	I-15 SB over 800 S	1
200	PC	Ramp NW over 400 S	3
212	PC	I-15 NB over 200 S	3
214	PC	I-15 SB over 200 S	3
220	PC	I-15 NB over N Temple	3
222	PC	I-15 SB over N Temple	3
224	PC	I-15 NB over 300 N	5
226	PC	I-15 SB over 300 N	5
12002	PC	I-15 SB CD over 2100 S	1

Query 6 – All SPC Bridge Structures

RFP Bridge Number	Type of Structure	Location	CSIN
8	SPC	I-15 NB over 9000 S	1
10	SPC	I-15 SB over 9000 S	1
20	SPC	I-15 NB over 7200 S	1
22	SPC	I-15 SB over 7200 S	1
30	SPC	I-15 NB over UTA RR	1
32	SPC	I-15 SB over UTA RR	1
54	SPC	I-15 NB over 5300 S	1
56	SPC	I-15 SB over 5300 S	1
64	SPC	I-15 NB over 4500 S	1
66	SPC	I-15 SB over 4500 S	1
74	SPC	I-15 NB over 3300 S	1
76	SPC	I-15 SB over 3300 S	1

Query 7 – All SC Bridge Structures

RFP Bridge Number	Type of Structure	Location	CSIN
26	SC	I-15 NB over UPRR	5
27	SC	I-215 EB to I-15 SB Ramp over UPRR	3
28	SC	I-15 SB over UPRR	5
50	SC	I-15 NB over 5900 S	3
52	SC	I-15 SB over 5900 S	3
230	SC	Ramp SW over 200 S	5

Query 8 – All SPT Bridge Structures

RFP Bridge Number	Type of Structure	Location	CSIN
23	SPT	I-15 NB / I-215 WB Ramp over 7200 S	5
112	SPT	I-80 WB (Ramp) To SR 201 WB over UTA RR	5
114	SPT	I-80 WB over UTA RR	3
116	SPT	I-80 EB over UTA RR	3
145	SPT	I-15 NB over Andy Ave	3
147	SPT	I-15 SB over Andy Ave	3
149	SPT	I-15 NB CD over Andy Ave	1
151	SPT	I-15 SB to I-80 EB over Andy Ave	3
174	SPT	I-15 NB over 900 S	3
176	SPT	I-15 SB over 900 S	3
196	SPT	I-15 NB over 400 S	5
198	SPT	I-15 SB over 400 S	5
202	SPT	Ramp ES over 400 S	5
216	SPT	I-15 NB over S Temple	5
218	SPT	I-15 SB over S Temple	5
702	SPT	I-15 SB CD over 7200 S	5

Query 9 – All Bridges with Longitudinal Cracking

RFP Bridge Number	Type of Structure	Location	Longitudinal Cracking	CSIN
29	PC	I-215 EB to 7200 S / I-15 SB CD Ramp over UTA RR	Yes	1
50	SC	I-15 NB over 5900 S	Yes	3
60	PC	I-15 NB over 4800 S	Yes	3
62	PC	I-15 SB over 4800 S	Yes	2
140	PC	I-15 NB over 2100 S	Yes	2
142	PC	I-15 SB CD over 2100 S	Yes	1
168	PC	I-15 NB over 500 W and UPRR	Yes	3
200	PC	Ramp NW over 400 S	Yes	3
220	PC	I-15 NB over N Temple	Yes	3
222	PC	I-15 SB over N Temple	Yes	3
226	PC	I-15 SB over 300 N	Yes	5

Query 10 – All Bridges with Integral Abutments

RFP Bridge Number	Type of Structure	Location	Abutment Type	CSIN
3	PC	I-15 SB over 10000 S	Integral	1
3.5	PC	I-15 NB over 10000 S	Integral	0
12	PC	I-15 NB over Wasatch St (8000 S)	Integral	1
14	PC	I-15 SB over Wasatch St (8000 S)	Integral	1
16	PC	I-15 NB over 7800 S (Center St)	Integral	2
18	PC	I-15 SB over 7800 S (Center St)	Integral	1
29	PC	I-215 EB to 7200 S / I-15 SB CD Ramp over UTA RR	Integral	1
50	SC	I-15 NB over 5900 S	Integral	3
52	SC	I-15 SB over 5900 S	Integral	3
60	PC	I-15 NB over 4800 S	Integral	3
62	PC	I-15 SB over 4800 S	Integral	2
70	PC	I-15 NB over UPRR	Integral	5
72	PC	I-15 SB over UPRR	Integral	5
112	SPT	I-80 WB (Ramp) To SR 201 WB over UTA RR	Integral	5
114	SPT	I-80 WB over UTA RR	Integral	3
116	SPT	I-80 EB over UTA RR	Integral	3
138	PC	I-15 NB CD over 2100 S	Integral	1
140	PC	I-15 NB over 2100 S	Integral	2
142	PC	I-15 SB CD over 2100 S	Integral	1
144	PC	I-15 SB over 2100 S	Integral	2
145	SPT	I-15 NB over Andy Ave	Integral	3
147	SPT	I-15 SB over Andy Ave	Integral	3
148	PC	I-15 NB CD over 1700 S	Integral	1
150	PC	I-15 SB CD over 1700S	Integral	2
151	SPT	I-15 SB to I-80 EB over Andy Ave	Integral	3
152	PC	I-15 NB over 1700 S	Integral	1
153	PC	I-15 SB CD over Andy Ave	Integral	1
154	PC	I-15 SB over 1700 S	Integral	2
155	PC	I-15 SB CD to I-15 SB over Andy Ave	Integral	1
156	PC	I-15 900S-A over 1300 S	Integral	1
158	PC	I-15 SB CD over 1300 S	Integral	2
160	PC	I-15 NB over 1300 S	Integral	2
162	PC	I-15 SB over 1300 S	Integral	2
168	PC	I-15 NB over 500 W and UPRR	Integral	3
170	PC	I-15 SB over 500 W and UPRR	Integral	3
200	PC	Ramp NW over 400 S	Integral	3
212	PC	I-15 NB over 200 S	Integral	3
214	PC	I-15 SB over 200 S	Integral	3
220	PC	I-15 NB over N Temple	Integral	3
222	PC	I-15 SB over N Temple	Integral	3
224	PC	I-15 NB over 300 N	Integral	5
226	PC	I-15 SB over 300 N	Integral	5
12002	PC	I-15 SB CD over 2100 S	Integral	1

Query 11 – All Bridges with Semi-Integral Abutments

RFP Bridge Number	Type of Structure	Location	Abutment Type	CSIN
8	SPC	I-15 NB over 9000 S	Semi-Integral	1
10	SPC	I-15 SB over 9000 S	Semi-Integral	1
20	SPC	I-15 NB over 7200 S	Semi-Integral	1
22	SPC	I-15 SB over 7200 S	Semi-Integral	1
23	SPT	I-15 NB / I-215 WB Ramp over 7200 S	Semi-Integral	5
30	SPC	I-15 NB over UTA RR	Semi-Integral	1
32	SPC	I-15 SB over UTA RR	Semi-Integral	1
54	SPC	I-15 NB over 5300 S	Semi-Integral	1
56	SPC	I-15 SB over 5300 S	Semi-Integral	1
64	SPC	I-15 NB over 4500 S	Semi-Integral	1
66	SPC	I-15 SB over 4500 S	Semi-Integral	1
74	SPC	I-15 NB over 3300 S	Semi-Integral	1
76	SPC	I-15 SB over 3300 S	Semi-Integral	1
180	PC	I-15 NB over 800 S	Semi-Integral	3
182	PC	I-15 SB over 800 S	Semi-Integral	1
702	SPT	I-15 SB CD over 7200 S	Semi-Integral	5

Query 12 – All Bridges with Expansion Abutments

RFP Bridge Number	Type of Structure	Location	Abutment Type	CSIN
26	SC	I-15 NB over UPRR	Expansion	5
27	SC	I-215 EB to I-15 SB Ramp over UPRR	Expansion	3
28	SC	I-15 SB over UPRR	Expansion	5
149	SPT	I-15 NB CD over Andy Ave	Expansion	1
174	SPT	I-15 NB over 900 S	Expansion	3
176	SPT	I-15 SB over 900 S	Expansion	3
196	SPT	I-15 NB over 400 S	Expansion	5
198	SPT	I-15 SB over 400 S	Expansion	5
202	SPT	Ramp ES over 400 S	Expansion	5
216	SPT	I-15 NB over S Temple	Expansion	5
218	SPT	I-15 SB over S Temple	Expansion	5
230	SC	Ramp SW over 200 S	Expansion	5

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11.3 Appendix C - Summary Table of Non I-15 Reconstruction Project Bridges

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Bridge Deck Survey of Non I-15 Reconstruction Project Bridges

Bridge Location	Inspection Date	Girder Type	Deck Type	Crack Ranking	Comments
I-15 NB over 12300 South	3/17/2004	Single-span spliced, post-tensioned precast concrete	Cast-In-Place	4	Extensive diagonal restraint cracking at both ends. Some longitudinal cracking. Only opened to traffic a few months.
I-15 NB over Highland Drive in Bluffdale	3/17/2004	3-Span precast concrete	Cast-In-Place	0	No visible deck cracks from the bridge underside.
I-15 SB over Highland Drive in Bluffdale	3/17/2004	3-Span precast concrete	Cast-In-Place	0	No visible deck cracks from the bridge underside. Deck may have been replaced at some point.
I-15 SB over SR 92	3/17/2004	3-Span precast concrete	Cast-In-Place	1	Apparent integral abutments and construction joint in deck. Minor diagonal restraint cracking at bridge ends and a few transverse cracks.
I-15 NB over SR 92	3/17/2004	3-Span precast concrete	Cast-In-Place	1	A few transverse cracks and a few diagonal restraint cracks at the bridge ends.
Railroad over Bangerter Highway just East of Jordan River	3/17/2004	Closely spaced 3-span steel girders	Cast-In-Place	1	Minor cracking and no regularly spaced transverse shrinkage cracks.
Bangerter Highway EB over 3600 West	3/17/2004	Single-span steel	Cast-In-Place	3	Large skew bridge. Evidence of epoxy treatments for some cracks.
Bangerter Highway WB over 3600 West	3/17/2004	Single-span steel	Cast-In-Place	5	Many cracks due to the large skew.
Bangerter Highway NB over 11800 South	3/17/2004	Single-span precast concrete	Cast-In-Place	2	Diagonal restraint cracking near abutments. Some transverse cracking.
Bangerter Highway SB over 11800 South	3/17/2004	Single-span precast concrete	Cast-In-Place	2	Diagonal restraint cracking near abutments. Some transverse cracking.

I-215 EB over Redwood Road (south)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	1	Minor transverse cracks and some diagonal restraint cracking at bridge ends.
I-215 WB over Redwood Road (south)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	3	More transverse cracks than EB bridge and some diagonal restraint cracking at bridge ends. Also has some limited map cracking on deck underside.
I-215 NB over 5400 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	1	Very minimal amount of cracking.
I-215 SB over 5400 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	1	Very minimal amount of cracking.
I-215 NB over 4700 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	5	Many transverse cracks.
I-215 SB over 4700 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	3	Less transverse cracking than NB bridge.
I-215 NB over 3800 South (west)	3/17/2004	Single-span steel	Cast-In-Place	3	Some transverse and diagonal cracking
I-215 SB over 3800 South (west)	3/17/2004	Single-span steel	Cast-In-Place	1	Minor transverse and diagonal restraint cracking.
I-215 NB over 3500 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	1	Some minor transverse cracking and very limited diagonal restraint cracking. Bridge ends appear to be free from restraint.

I-215 SB over 3500 South (west)	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	1	Some minor transverse cracking and very limited diagonal restraint cracking. Bridge ends appear to be free from restraint.
Ramp 3500 South EB to I-215 NB over 3500 South	3/17/2004	2-span curved steel girders	Cast-In-Place	3	Some apparent patchwork on deck underside. North span has many transverse cracks and south span has very limited transverse cracks a few diagonal cracks.
SR 201 EB over Redwood Road	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	3	Girders have rust and need paint badly. Some transverse and diagonal cracking. Exposed deck rebar and extensive cracking due to spreading corrosion.
SR 201 WB over Redwood Road	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	3	Girders have rust and need paint badly. Some transverse and diagonal cracking. Exposed deck rebar and extensive cracking due to spreading corrosion.
I-215 NB over 1700 South (west)	3/17/2004	Precast concrete	Cast-In-Place	4	Many diagonal restraint cracks and some transverse cracks near bridge ends. No visible cracking near center span.
I-215 SB over 1700 South (west)	3/17/2004	Precast concrete	Cast-In-Place	4	Many diagonal restraint cracks and some transverse cracks near bridge ends. No visible cracking near center span.
I-215 NB over California Avenue (west)	3/17/2004	Long single-span steel	Cast-In-Place	5	Many transverse cracks across length of bridge. Some diagonal restraint cracking at bridge ends. It appears that water is coming through the deck and running down the girders and dripping from the bottom flanges of the girders.
I-215 SB over California Avenue (west)	3/17/2004	Long single-span steel	Cast-In-Place	5	Many transverse cracks across length of bridge. Some diagonal restraint cracking at bridge ends. It appears that water is coming through the deck and running down the girders and dripping from the bottom flanges of the girders.
Ramp I-80 EB to North Temple over I-80	3/17/2004	Multi-span steel girders	Cast-In-Place	5	Many, many transverse cracks closely spaced over entire bridge length.

Road to Cargo Shipping over Inbound Airport Traffic	3/17/2004	3-span steel	Cast-In-Place	5	Many cracks all over bridge deck.
Road to Cargo Shipping over Outbound Airport Traffic	3/17/2004	3-span steel	Cast-In-Place	5	Many cracks.
I-215 NB over Ramp from I-80 EB to I-215 NB	3/17/2004	2-span steel	Cast-In-Place	5	Many transverse cracks across full length of bridge.
I-215 SB over Ramp from I-80 EB to I-215 NB	3/17/2004	2-span steel	Cast-In-Place	5	Many transverse cracks across full length of bridge.
I-215 NB over 700 North (west)	3/17/2004	Single-span precast concrete	Cast-In-Place	2	Considerable amount of diagonal restraint cracking near bridge ends.
I-215 SB over 700 North (west)	3/17/2004	Single-span precast concrete	Cast-In-Place	2	Diagonal restraint cracking near bridge ends.
1700 North over I-215 (west)	3/17/2004	3-Span precast concrete	Fluted Metal Decking	0	No visible cracking through the metal decking.
I-15 NB over Center Street in Woods Cross	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	5	The steel girders are very shallow. Many transverse cracks across full length of bridge.
I-15 SB over Center Street in Woods Cross	3/17/2004	Single-span steel girders with small cantilever at ends	Cast-In-Place	5	The steel girders are very shallow. Many transverse cracks across full length of bridge.
I-15 NB over Main Street in Woods Cross	3/17/2004	Single-span steel	Cast-In-Place	5	Bridge has a very large skew and many transverse cracks.

I-15 SB over Main Street in Woods Cross	3/17/2004	Single-span steel	Cast-In-Place	5	Bridge has a very large skew and many transverse cracks.
I-15 NB over 2600 South in Bountiful	3/17/2004	3-span concrete	Cast-In-Place	1	Very few cracks in old deck which may have been cast monolithically with the girders. Recently, New steel girder sections of the bridge were placed between the NB and SB bridges to accommodate additional traffic lanes. These new deck sections are severely cracked everywhere. Corrosion appears to be moving at a fast rate.
I-15 SB over 2600 South in Bountiful	3/17/2004	3-span concrete	Cast-In-Place	1	Very few cracks in old deck which may have been cast monolithically with the girders.
I-15 NB over 500 South in Bountiful	3/17/2004	3-span concrete	Cast-In-Place	1	Similar situation to 2600 South. Old bridge deck has few cracks while the new steel girder additions have many, many cracks.
I-15 SB over 500 South in Bountiful	3/17/2004	3-span concrete	Cast-In-Place	1	Similar situation to 2600 South. Old bridge deck has few cracks while the new steel girder additions have many, many cracks.
Parrish Lane over I-15 in Centerville	3/17/2004	2-span steel	Cast-In-Place	1	A few minor transverse cracks near center span and no visible diagonal restraint cracking at bridge ends.
Glovers Lane over I-15 in Farmington	3/17/2004	3-span steel	Cast-In-Place	3	Some transverse cracks.
I-215 NB over Holliday Blvd. (east)	7/7/2003	Single-span steel	Fluted Metal Decking	2	Diagonal cracks at the bridge ends are showing through the metal decking in the form of rust lines. Deck membrane overlay now covering top deck surface.
I-215 SB over Holliday Blvd. (east)	7/7/2003	Single-span steel	Fluted Metal Decking	2	Diagonal cracks at the bridge ends are showing through the metal decking in the form of rust lines. Less visible cracking than NB bridge. Deck membrane overlay now covering top deck surface.

I-215 NB Exit Ramp over Holliday Blvd. (east)	7/7/2003	Single-span steel	Fluted Metal Decking	1	Some transverse cracks showing through decking as rust lines.
I-215 NB over Tolcate Lane (east)	7/7/2003	Single-span precast concrete	Fluted Metal Decking	0	No visible cracking through the metal decking.
I-215 SB over Tolcate Lane (east)	7/7/2003	Single-span precast concrete	Fluted Metal Decking	0	No visible cracking through the metal decking.
Union Park Blvd. NB to I-215 WB Ramp over I-215	7/7/2003	Multi-span curved steel girders	Cast-In-Place	5	Many closely spaced transverse cracks across entire bridge length.
I-80 EB over 2000 East	7/8/2003	3-span precast concrete	Cast-In-Place	1	Minor cracking visible on the deck underside.
I-80 WB over 2000 East	7/8/2003	3-span precast concrete	Cast-In-Place	2	Some transverse, diagonal, and longitudinal cracking near bridge ends. Some exposed rebar due to years of water exposure.

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